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Sir Hubert Walker, Member, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Airport Paper No. 21

"The Organization of Airport Construction"

by

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SYNOPSIS

A brief account of the magnitude of the work involved at London Airport is given and the methods of carrying out the various operations are then described. The way in which earthworks were organized and the types of earth-moving plant used are discussed. The methods used for the compaction of the gravel sub-base and the preparation of formation are also described.

Four different types of major concrete-mixing plant were used for the high-quality paving concrete and these are described with drawings, and the relative advantages considered. An account of the method of placing concrete is given. The way in which the ducting and stormwater drainage systems were constructed is briefly described.

The testing and control of materials was a very important part of the organization and the work of the Contractor's site laboratory is described. Its principal duties were the keeping of weather records, testing aggregate samples, making and keeping records of concrete aggregate deliveries, calibration of concrete set-ups, testing fill for density, etc.; some further figures on concrete test cubes results are given and the method by which the problem of excess moisture contents in aggregates at time of delivery was overcome, is described.

The Contractor's site organization including the machinery of welfare and labour relations is outlined, and the respective merits of decentralization are discussed.

Introduction

Since it would be difficult to generalize on the subject of the organization of airport construction, the following Paper is an account of the problems encountered and the methods used in organizing the construction of the main runways and ancillary undertakings at what can probably claim to be the largest airport in the world, and certainly one of the greatest civil engineering projects carried out in Great Britain in recent years, namely, the London Airport at Heathrow. Various aspects of this undertaking have been very interestingly described in previous Papers,[1, 2, 3] and inevitably there is some duplication in the background details given in this Paper. Nevertheless it is hoped that a description of the Airport from the point of view of the organization of the operations involved in its construction will be of interest and help to round off the picture given by previous Papers, which necessarily confined themselves to more technical aspects. Many of the methods used have now become commonplace, but it must be remembered that when Heathrow was first started this was by no means the case and much pioneering and experimental work had to be carried out before they were evolved.

The site selected for the construction of the airport consisted of flat agricultural land, unwooded and intersected by a number of roads. The ground strata consisted of 12 inches of topsoil, above a layer of friable clay, called brickearth, ranging in thickness from 2 to 4 feet; underneath this was a bed of gravel, 10 to 30 feet thick, lying on blue London clay. The water table was generally 4 to 6 feet beneath the surface of the ground (*Fig. 1*). Near the northern boundary of the site, the underlying gravel had been worked commercially for a number of years and the resulting water-filled pits, which were of considerable area, formed a special problem in the early stages of construction.

Limitations imposed by design and by the necessity for keeping as much land as possible under cultivation until the last possible moment meant that construction had to be split into a number of phases (Fig. 2, Plate 1). Whilst these phases overlapped to the extent that continuity was not seriously affected, there is no doubt that they were the major factor in determining the programming of the work and, had it been practicable to plan work as a whole, greater economies in both method and time would have been possible. On the other hand, the phased programme of work allowed a continual evolution of improvement in method which otherwise could not have been so easily secured.

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[1] G. Graham and F. R. Martin, "Heathrow. The Construction of High-grade Quality Concrete Paving for Modern Transport Aircraft." J. Instn Civ. Engrs, vol. 26, No. 6, p. 117 (April 1946).

[2] J. K. Fisher, A. Goode, and C. E. Docker, "Some Problems in the Design and Construction of Large Airfields." Airport Paper No. 3, Instn Civ. Engrs, 1946.

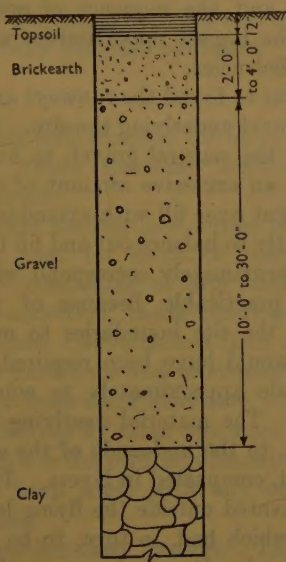
[3] H. Smith, "Gravel Compaction and Testing, and Concrete Mix Design at London Airport." Proc. Instn Civ. Engrs, Part II, vol. 1, p. 1 (Feb. 1952).

Starting in May 1944, the work continued at a varying pace for the following 7½ years, during which period some of the major quantities involved were as listed below :—

Excavation	16,000,000 cubic yards
Compacted gravel fill	3,200,000 " "
Concrete	1,750,000 " "
Stormwater drains	91 miles
Lighting ducts	165 "

It is rarely that such large quantities are involved in any one scheme carried out, and it can be readily understood that the cumulative experience which it was possible to gain as time passed resulted in the attainment of an unusually high degree of skill by all concerned with the construction.

Fig. 1



Typical Section through Ground Strata

This applied particularly to the workmen, many of whom had 6 or 7 years of service at the same job and became highly expert in the techniques which were evolved.

A further advantage secured by the length of time over which the contract extended lay in the opportunity afforded to take the maximum advantage of adjusting different operations to seasonal variations in weather. For example, earthworks can only be carried out very inefficiently during the 4 months from November to February, and it was

found that the proper compaction of gravel fill was practically impossible during these months, because of the difficulty of keeping the moisture content within reasonable limits. During the winter of 1944–45, gravel compaction was insufficiently far advanced and the only way in which progress could be maintained was by substituting weak concrete. In succeeding years it was possible to programme for accelerating earthworks and gravel fill sufficiently so that work on these items could be suspended during the 4 winter months. Much waste of effort and risk of bad workmanship was avoided in this way, but only because it was possible to think ahead in terms of years rather than months.

Earthworks

Although all the individual items of earthworks were quite straightforward, consisting mostly of shallow surface digging, the quantities involved were so large and the sequence of operations so complicated that it was here that the largest opportunities were afforded for saving effort and avoiding inefficiencies.

The general object was to found the runways and other paved surfaces on the natural bed of gravel underlying the site. Since several feet of soil and brickearth overlay the natural gravel, to have founded directly on this would have meant an excessive amount of excavation in regrading and a large surplus of cut over fill with attendant problems of disposal. To have attempted exactly to balance cut and fill by making the underside of the concrete raft approximately correspond with the original average ground level was not practicable, because of the non-availability of sufficient gravel within the site boundaries to make the natural gravel up to the level which would have been required. In practice, the new surface levels were made approximately to coincide with the original average ground levels. The material overlying the natural gravel was dug away and the level, to the underside of the concrete slab, was made up with imported gravel, compacted in layers. This imported gravel was obtained from pits excavated outside the flying lanes and safety margins of the paved surfaces, which had, in turn, to be filled with the material which the gravel replaced. It can be seen, therefore, that to avoid wasteful double-handling and to keep lengths of haul to the minimum, considerable scope for careful planning was afforded. The presence of the old gravel pits afforded a useful outlet for fill to start the sequence of operations going, and the fact that it could be calculated that ultimately a volume approximately equal to the volume of concrete paving would be surplus, and therefore have to be taken off site, also afforded a safety factor which was used to prevent double-handling. A typical earth-movement chart for a section of the work is shown in Fig. 3, Plate 1.

The information on which these earth-movement charts were based was necessarily subject to considerable inaccuracies. The principal ones

derived from the lack of certainty as to the volume and quality of gravel which would be obtained from any particular borrow-pit. This uncertainty was the result of several factors, namely, the considerable local fluctuations in the thickness of the gravel bed; the variations in the level of the surface of the gravel bed, and hence of the depth to which surface excavation had to be taken—which, in shallow digging over a very large superficial area, could produce very considerable errors; and variations in the bulkage of the earth refilling to the gravel borrow-pits, caused by a tendency to compact to a greater degree than required by the specification, owing to the use of very heavy earth-moving plant. The second variable was kept to the minimum by sinking frequent trial holes to ascertain the gravel surface level, and the first and third variables became less important as experience of the site was gained. Although it was possible to work surprisingly closely to the earth-movement charts, the presence of these variables necessitated constant revision to secure maximum efficiency.

After preparation of the chart, the type and quantity of earth-moving equipment was decided from the mass-haul data. The types of equipment generally used were :—

(1) Tracked tractors and scrapers for short hauls of no more than 200 yards. These machines were also generally used for stripping topsoil.

(2) Elevating graders loading into lorries or bottom-dump wagons. These were used for hauls of intermediate distance; the friable earth, containing no large stones, with large areas over which to manoeuvre, made conditions ideal for the use of these machines. Nevertheless their extreme sensitiveness to wet ground conditions and proneness to breakdown detracted considerably from their efficiency. Very large quantities of earth were moved by these machines at the peak of the programme and it is doubtful if, without them, sufficient progress could have been made with any other type of equipment then available.

(3) Scrapers and rubber-tired tractors. These were used for hauls of between 200 and 500 yards. They need push-assistance by bulldozers in loading, because the tractor tires do not give sufficient grip. Generally they were found to give a much lower rate of efficiency than other methods and their use was restricted to the minimum.

(4) Lorries loaded by excavators were employed for the longer hauls and for the large quantities of material which had to be taken outside the site. They were also used for excavating gravel from the borrow-pits. Owing to the shallowness of the excavation and its soft nature, small machines with a fast cycle of operation were found to be more economical than

TABLE 1—EXAMPLE OF WEEKLY MACHINE-EFFICIENCY ANALYSIS

Machine No.	Hours				Production			
	Total shift	Available working	Broken down	Lost by weather	Standard per hour	Standard output available (Total)	Actual total	Percentage efficiency
Runway 4 North.								
15B D/L 15.36310	47½	47½	—	nil	42	1,995	—	—
15B D/L 9.27692	49	49	—	nil	42	2,058	—	—
19RB D/L W.44	43½	43½	—	nil	55	2,393	—	—
19RB D/L W.114	49	44	5	nil	55	2,420	—	—
24RB D/L 6972	12½	12½	—	nil	74	925	—	—
33RB D/L 4697	48	48	—	nil	114	5,472	—	—
Totals	249½	244½	5	nil	67	15,263	12,942	85·4
Concreting								
Main set-up	49	47	2	—	90	4,230	2,861	67·6
Transport	488	488	—	—	9	4,392	2,861	65·1
Auxiliary set-up	49	48½	½	—	52·5	2,546	1,980	77·8
Transport	451¼	451¼	—	—	5·9	2,662	1,980	74·4

larger machines. Draglines and crowd shovels were used for surface excavation and draglines for the borrow-pits.

Standard performances were assessed for all machines, and outputs, measured weekly, were compared with these to determine the efficiency obtained. These records were tabulated on forms (Table 1) and careful investigation was undertaken for any machine or group of machines which showed an unduly low percentage of efficiency.

An exceptional problem was encountered at the commencement of operations when the old existing gravel pits had been pumped clear of water. It was found that years of gravel washing had resulted in the accumulation of a layer of silt, 10 feet thick in places, which had to be removed before the pits could be refilled. When first exposed the silt was of a semi-liquid consistency, and was very difficult to deal with; the silt was removed, by draglines standing on the banks, from around the perimeter of the pits, and the channels so formed were led into deep sumps which were pumped continuously. After a week or two it was usually possible to excavate the silt dewatered in this way with crowd shovels and draglines working on the bottom. Subsidiary rubble drains were led to local hollows in the bottoms of the pits, and the drainage systems so formed were pumped continuously until refilling had been completed, so that the water table was kept down and all filling could be carried out in the dry. About 800,000 cubic yards of silt had to be dealt with altogether and removed off site.

More than 1,000,000 cubic yards of surplus excavation was carted off site and dumped into old gravel pits, and a large area of land was thus reclaimed. The rich topsoil overlying the site was throughout separated very carefully from other excavation; material not needed for re-soiling graded areas was carted off site and spread on the areas of hitherto infertile land in the vicinity of the Airport.

Access Roads

Owing to the large proportion of earth movement done by lorry and the necessity for conveying materials to, and concrete away from, the central batching plants, internal site transport was always very heavy. This is illustrated by the fact that approximately 3,200,000 gallons of petrol were used for site haulage, representing more than 25,000,000 miles travelled by the lorries. In the circumstances, it was apparent that considerable economies in time and in wear and tear on lorries could be made by careful attention to site access roads. A great advantage was gained from the existing system of roads which crossed the site, but many of these had to be widened and strengthened to stand up to the demands put upon them. In addition, many thousands of square yards of new concrete access roads, 20 feet wide and 6 inches thick, were laid. The

internal road system is shown in Fig. 2, Plate 1. The lay-out was largely determined by the fact that the Airport became operational early in 1946 and, during the greater part of the contract, therefore, site transport had to be very carefully controlled so that there was no danger to aircraft. Alternative routes had always to be ready so that whenever any shift of wind necessitated another runway being brought into operation, it could be cleared of site traffic immediately with the least possible delay to work. This was achieved by close liaison between the transport supervision and flying control and it is remarkable that during the 6 years in which these conditions existed, no serious difficulties ever arose.

Gravel Compaction

The gravel used in making-up from the surface level of the existing gravel bed to the underside of the concrete slab had to be compacted to a minimum density of 95 per cent of Proctor density, equivalent to 128 lb. per cubic foot.* To achieve the required result, the grading had to be such that the quantity of material passing the No. 100 sieve was not less than 15 per cent and not more than 30 per cent of the quantity of material passing the $\frac{3}{16}$-inch sieve ; if the silt content was less than this the gravel was too coarse to bind properly and give satisfactory results. Generally the gravel excavated from site borrow-pits complied with grading requirements, although there were slight variations in silt content up to the permitted limit ; gravel with the higher silt content was, of course, more easy to compact satisfactorily and required less rolling.

Great care had to be taken, when excavating gravel from site borrow-pits, to keep the water table down to below the level of the stratum. If that was not done a high proportion of fines was washed out during excavation and the gravel became useless for compaction. In opening up a borrow-pit, therefore, the first step, after stripping overburden, was to dig a deep sump through the gravel strata into the underlying clay ; gravel from those sumps, which were often of considerable size, had to be rejected for compacting purposes. Pumps were then installed and only after the gravel for a considerable area around the sump had been completely de-watered was excavation commenced. Owing to the height of the water table and permeable nature of the strata, immense quantities of water had to be dealt with and organization of pumping was one of the major considerations in construction. Self-priming centrifugal pumps of 4 and 6 inches diameter were generally used, although for the initial pumping-out of the existing gravel pits, 12-inch pumps were employed. At one stage of construction, fifty pumps of various sizes were in use and the supervision required, on a 24-hour basis, to safeguard against breakdown was considerable. In the early stages of construction, water was led away through existing ditches, but when the permanent drainage system could be started

* Corrected from 126 lb. per cubic foot. *See* Discussion, pp. 32 and 37.

it was programmed so that the maximum use of it could be made for pumping operations.

The optimum moisture content of the gravel for satisfactory compaction was between 4 and 6 per cent, and it was generally found that this was the content which it retained naturally after the borrow-pit had been de-watered. After the gravel had been excavated and placed, therefore, it rarely needed any further treatment and could be rolled immediately. It sometimes happened that during very hot weather, or if rolling was not started quickly enough, the gravel became too dry, in which case it was sprayed with water from tankers. Control of the rate of application was obtained by the number of passes given at a determined speed, the rate of spray being known. The water was pumped through the spray bars so that the rate of spray was not affected by the lowering of the head of water in the tank. The water tankers, which had a capacity of 1,100 gallons, were specially designed for the contract; not only could they be used for increasing the water content of the gravel, but they also had side spraying nozzles attached for curing concrete and the pump could be reversed, so that they could be used for pumping out local pockets of water after heavy rains.

Gravel was placed in layers so that the final consolidated thickness of earth layer was not more than 9 inches. Careful records were kept and, where fills were more than one or two layers thick, levels were taken on the top of each layer, after compaction, as a check that the maximum thickness was not being exceeded. Compaction was carried out with sheepsfoot and rubber-tired rollers: sheepsfoot rollers were pulled in tandem by tractors and had a bearing pressure of 350 lb. per square inch on each foot. Rubber-tired rollers had a total weight of approximately 12 tons loaded on sixteen tires—eight at the back and eight at the front. Some of these rollers were of the wobbly-wheel type, using aircraft-type tires without treads, others had fixed wheels and ordinary heavy-duty lorry tires. It was not found that one type had any particular superiority over the other. It was at first thought that the best results would be obtained with the sheepsfoot rollers, the rubber-tired rollers being used only for sealing-off the loose material left on the surface by the former. Results were not very satisfactory because, owing to the non-cohesive nature of the material, the sheepsfoot roller would not "walk out" properly and even after nine or ten passes only the bottom few inches of the layer had been compacted. Another very serious difficulty was that the deep indentations left by the roller trapped rainwater and, after a heavy rainstorm, made it necessary to cease operations altogether until the layer had dried out, which might take several days. On the other hand, the sealing operation carried out by the rubber-tired roller was very successful and, after a number of experiments, it was found that the quickest and most economical method of obtaining the desired result was to give two passes with the sheepsfoot roller, to make sure of compacting

the bottom of the layer, and to complete with the rubber-tired roller until the density was obtained. Generally, in the case of gravel with a normal silt content, four to six passes were sufficient.

In places where access was too restricted for the free operation of the rubber-tired roller, 10-ton smooth-wheel rollers were used and the thickness of layer was reduced to 4 inches.

Density tests were taken on every layer after rolling and it was soon found that it was vital to progress that the time taken for these tests should be reduced to the minimum, or gravel-placing operations were held up. A number of 10-cwt vans were therefore equipped as mobile field laboratories and tests were taken and checked, and the results worked out on the spot. Operations were zoned so that placing, compacting, and testing could be carried on simultaneously and the placing operation was always kept sufficiently far ahead so that, should a test fail and more rolling be necessary, placing was not interrupted.

The maximum depth of gravel fill to be placed and compacted was 25 feet, in the area where runways crossed the old gravel pits. Markers were placed on top of these deep fills and kept under observation for signs of settlement. No settlement was observed, nor have there been signs of any in the 6½ years that the particular runways have been in use.

Preparation of Formation

The aim in placing the final layer of gravel was to bring it so that, after compaction, it was as near as possible to the true formation level, and the degree of accuracy achieved was usually within $\pm$ 1 inch. Over most of the paved areas a sub-base layer of low-cement-content concrete, nominally 8 inches thick, was placed on top of the prepared formation. Heavy-duty steel roadforms were laid to true level on the compacted gravel surface, a dry cement-mortar being used for bedding to take out any irregularities in the surface and to ensure the stability of the forms. This cement mortar was afterwards removed at the same time as the forms.

Under summer conditions the gravel between the forms was trimmed by hand and rolled with a 50-cwt roller to give a general depth of 8 inches between forms. During the winter it was not possible satisfactorily to make up hollows in the existing surface with gravel and the aim was to disturb the surface as little as possible. The final layer was, therefore, left very slightly high and the minimum depth between forms of 7½ inches was permitted; any hollows were left untreated and an average depth of 8 inches was thereby obtained.

Concrete Mixing

Four main concrete mixes were in use at Heathrow. These were :—

(1) A nominal 5 : 1 by volume made with washed 1½″ down all-in

ballast, generally used for drain surrounds, manholes, etc. Referred to as Quality "B."

(2) A nominal 4·2 : 1 by volume concrete made with washed 3″ down stone and sand or premixed all-in ballast, used for duct surrounds, pits, etc. Referred to as Quality "E."

(3) A 6·2 : 1 by weight concrete made with $1\frac{1}{2}''$–$\frac{3}{4}''$, $\frac{3}{4}''$–$\frac{3}{8}''$, $\frac{3}{8}''$–$\frac{3}{16}''$ stone and sand. This was much the most important mix and nearly 1,000,000 cubic yards of it were laid in the 12″-thick top slab to all paved areas. Referred to as Quality "H."

(4) A nominal 12 : 1 by volume concrete made with $1\frac{1}{2}''$ down all-in ballast, used for the 8″-thick sub-raft, filling up holes in the formation, etc.

All concrete was mixed in central set-ups, sited to feed various sections of the work, and transported by lorry to the points of placing. For Qualities "B" and "E" and 12 : 1 concrete, continuous mixers of the Benford Regulus type were principally used. For Quality "H" concrete, because of the very large outputs required and the special requirements of the specification, much more complicated plants were necessary.

The main requirements of the specification which affected the design of the Quality "H" concrete plants were :—

(*a*) In order to economize in cement it was decided to weigh-batch all materials.

(*b*) To obtain a moisture content as uniform as possible all stone and sand had to be stockpiled for at least 24 hours before use, so that it should have time to drain out.

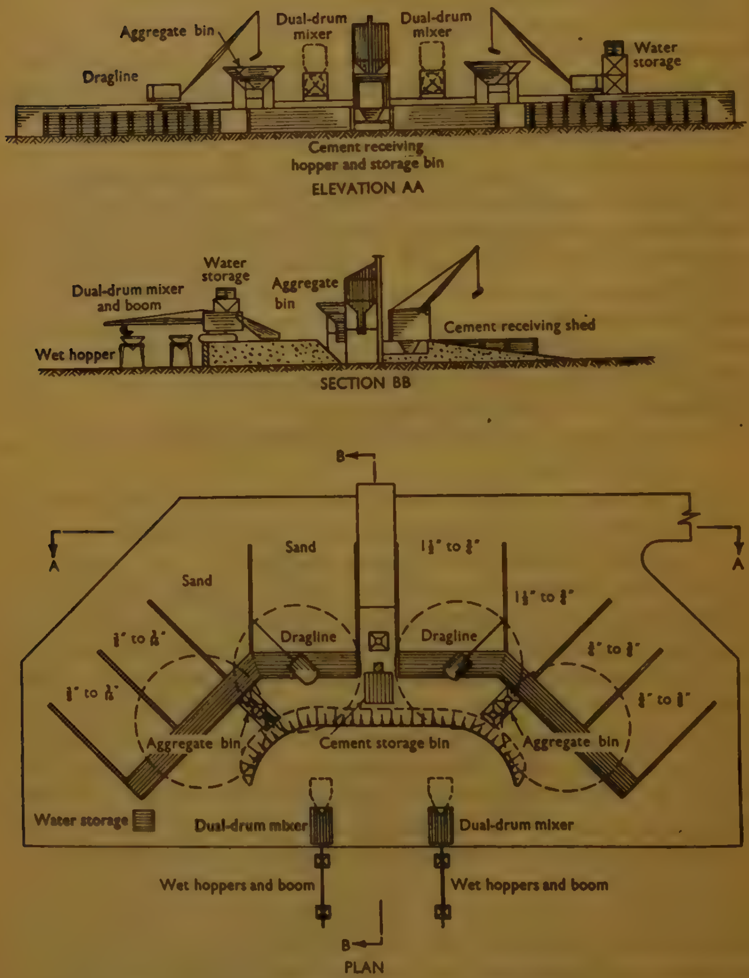
Since three different sizes of stone, as well as sand, were to be used, and in order to avoid double-handling, eight storage bins had to be incorporated. These bins had to have sufficient storage capacity for at least 2 days' aggregate supply, based on maximum output. They had to be constructed so that there was no possibility of contamination of the material in any way and so that there was adequate provision for drainage.

During the period of construction, four different arrangements of Quality "H" concrete plants were designed and erected and each arrangement was an improvement on the one preceding it, in both economy of erection and output. The four different types are shown in *Figs 4*.

Type 1

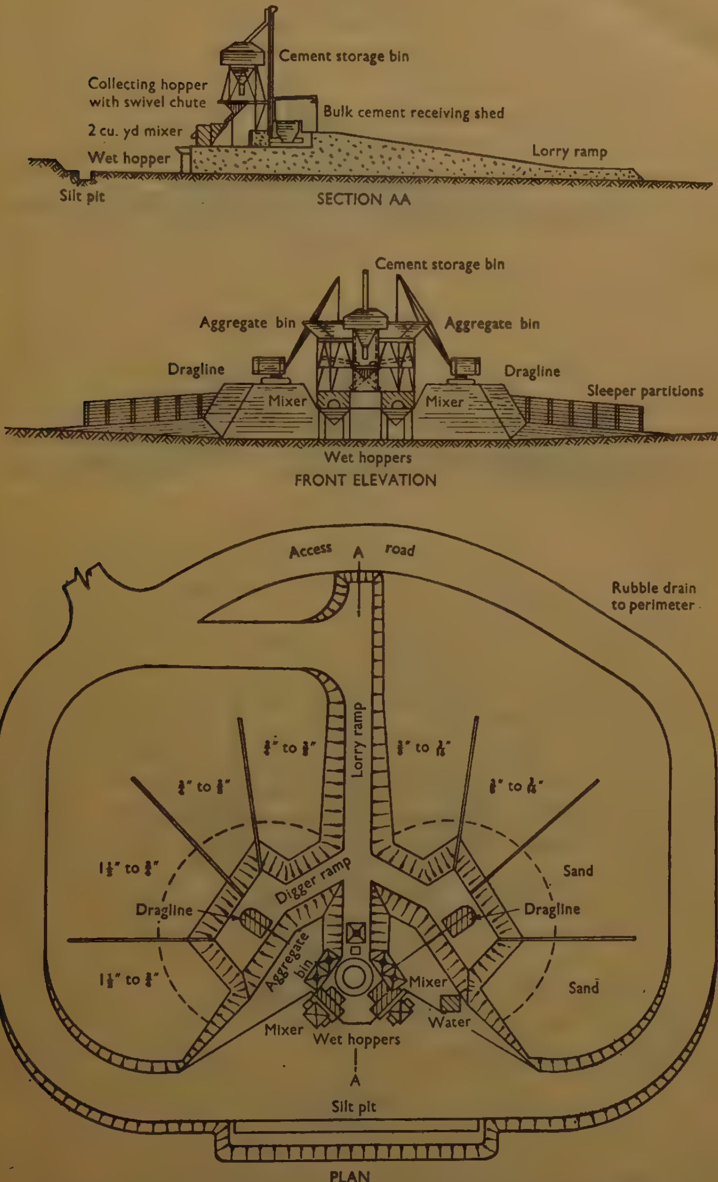
Two of these plants were erected and they were used for Phase 1 of construction—the main east–west runway and adjacent taxi track. The main feature was that the cement and aggregate batching plants were erected independently from the concrete-mixers and batched material was conveyed in dumpers between the two sections of the plant. This was necessary because the only suitable weigh-batching plants readily available in Britain at the time were two sets borrowed from the U.S. Army Corps. To obtain the outputs required, four 34E dual-drum Paver-type concrete-mixers were used and it was, therefore, necessary that each batching plant

Figs 4 (a)



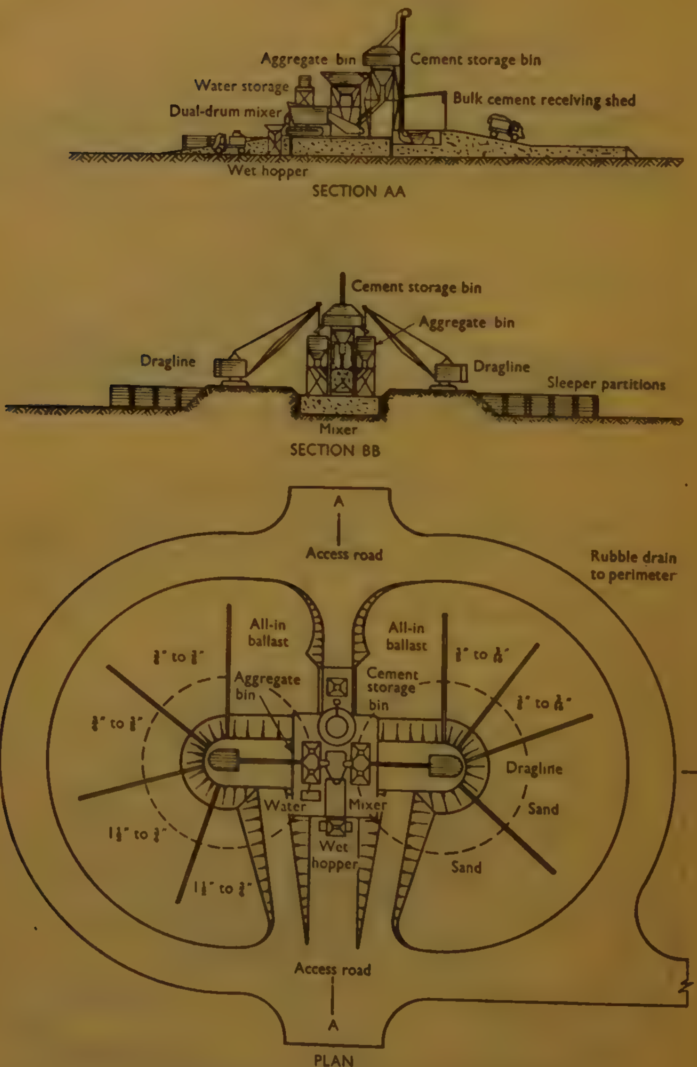
Type 1. Weigh-Batching Plant for High-Grade Concrete, utilizing Two Dual-Drum Mixers with Dumpers conveying Cement and Aggregates to Mixers

Figs 4 (b)



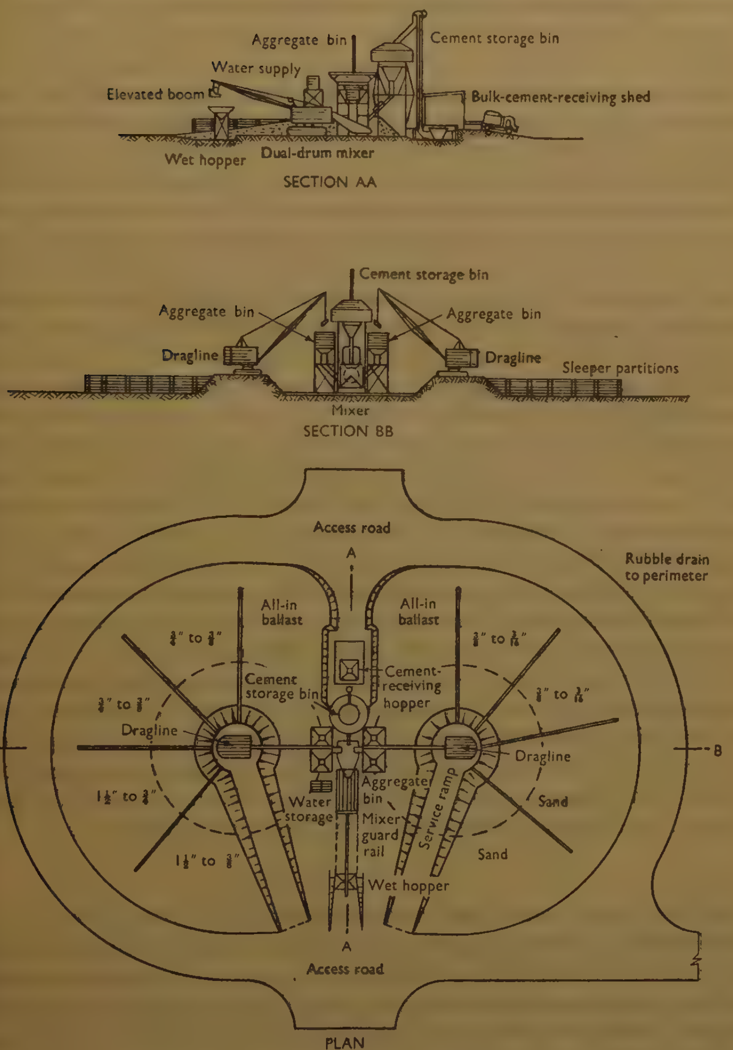
TYPE 2. TWIN 2-CUBIC-YARD WEIGH-BATCHING PLANT FOR HIGH-GRADE CONCRETE

Figs 4 (c)



Type 3. High-Level Single Dual-Drum Weigh-Batching Plant for High-Grade Concrete

Figs 4 (d)



TYPE 4. LOW-LEVEL CONCRETE WEIGH-BATCHING PLANT UTILIZING SINGLE DUAL-DRUM MIXER WITH ELEVATED BOOM

should feed two concrete-mixers. The plants were quite satisfactory in operation and outputs of up to 1,600 cubic yards per day from each set-up were obtained. The dumpers were always the worst source of trouble and a number of accidents, one fatal, occurred because of premature raising of the mixer hopper while the dumper was tipping its load.

Type 2

These plants were by far the most elaborate and expensive to erect. Two were used for Phase 2 (Runways 2 and 3). New batching plants were available by the time they were erected but the dual-drum mixers would not become free from Phase 1 in time for use. It was decided, therefore, to design set-ups incorporating one set of weigh-batching plant feeding by gravity into two 2-cubic-yard-capacity rotary-drum mixers with fixed hoppers. The whole plant was designed to be driven by electricity. Owing to the height from the ground of the point of discharge into the mixer hoppers the batching plant had to be raised high in the air and the arrangement was made practical only by the introduction of short conveyor belts to take the weighed aggregates from the batching hoppers to the central distributing hopper. Even so, the highest point of the set-up was 66 feet above ground level. Difficulty was at first experienced in getting the batched material to run freely from the distributing hopper into the mixers, but this was overcome by changing the design of the distributing hopper from a breeches chute with butterfly flap to a swivelling chute, and by fixing a small vibrating hammer on the back of the mixer hopper which came into operation as soon as the gate into the mixer was opened. After these modifications had been made the plant worked very successfully. It is interesting to note that in the case of these two plants very simple types of concrete-mixer were used, necessitating a heavy expenditure on the surrounding structure. All the other types erected employed much more complicated mixers, of much higher capital cost, but required a considerably smaller outlay on erection costs. Which type is preferable depends on the length of time the plant is to be in use and the total volume of concrete anticipated that it will be required to produce.

Type 3

Three of these plants were erected and they consisted of one set of weigh-batching plant feeding by gravity direct into the hopper of a 34E dual-drum Paver-type mixer. The essential part was a chute, working on a pivot, which the weight of aggregate falling from the batcher automatically caused to swing into the mixer hopper. These plants were comparatively easily erected and were very simple to operate.

Type 4

This was an improvement on Type 3 made possible by the introduction of an elevating boom for the discharge bucket of the Paver-type mixer.

This made it possible to place the mixer and batching plants very nearly at ground level, which resulted in a considerable saving in erection costs. One of these plants was erected.

Total aggregate storage capacity was 4,000 tons for the Type 1 plants and 2,500 tons for the other types. Each of the two aggregate weigh-batchers required for a plant dealt with two sizes of aggregate, being split into two compartments each holding 50 tons. Aggregate was conveyed from the storage bins up to the batching hoppers by dragline excavators. All cement was delivered in bulk and discharged into a hopper from which it was elevated to the 50-ton-capacity overhead storage hopper. Compressed air was fed into this hopper to overcome any tendency to arching when the cement was fed into the weighing hopper underneath. The scales of the weighing mechanisms were of the horizontal-beam type and were checked weekly with 50-lb. weights.

The consumption of water for mixing concrete was heavy and in addition a supply had to be provided for curing concrete, compacting operations, etc. In all, it was anticipated that peak daily consumption would reach 100,000 gallons. For the first phase of construction, water was drawn from one of the existing gravel pits on site but later a 6-inch main was brought from the south-eastern balancing reservoir outside the site. Water was pumped from the reservoir by means of a 6-inch pump, and two relay reservoirs of 125,000 gallons capacity each were constructed at intermediate points along the pipeline ; 6-inch pumps were installed at these points also and since stand-by pumps had to be provided in case of breakdown a total of six pumps was required. Branch lines to the main mixing plants were 4 inches in diameter and to the auxiliary plants, 2 inches. Overhead storage tanks were provided at all plants. For drawing-off water for curing and other purposes, stand-pipes were provided on the pipeline. The siting of mixing plants and lay-out of the water supply system is shown in Fig. 5, Plate 2. Great care had to be taken in designing levels for the water pipe-line to ensure that it did not foul future drains and ducts.

Concrete Placing

The lorries which were used for conveying concrete from the mixing plants to points of placing were ordinary 50-cwt tippers. The maximum distance concrete was hauled was approximately 3 miles (to one of the diversion roads round the site), and at no time was the length of haul found to have any effect on the quality of the concrete at the placing point. An experiment was at one time carried out by running a lorry, loaded with concrete, round the site and taking test cubes from the load at intervals. The results are given in Table 2.

At the 30-minute test, the vibration of the lorry was bringing some water to the top in the centre of the batch, although segregation of the

material itself was not noticeable until 3 hours 50 minutes had elapsed. The workability had changed very little after 1 hour 20 minutes, but after this a progressive deterioration began to set in.

Weather conditions for this test were very good but, even so, the results show a margin of safety sufficient to demonstrate that it is perfectly safe to use this method of distributing concrete on a large site.

Table 2

Weather during test.—Clear sky; sunny; cold wind, 13–18 m.p.h.; temperature, maximum 44° F., minimum 38° F.; humidity, 90 to 95 per cent.

Concrete.—6·2 : 1 by weight graded aggregates to ordinary Portland cement; water/cement ratio 0·48.

Time elapsed: hours	mins	Distance driven: miles	Strength of cubes in lb. per sq. in.: 7 days	Average	28 days	Average
	5	¼	3,700 3,020	3,360	6,570 6,660	6,615
	30	11	3,550 3,480	3,515	6,780 6,630	6,705
1	20	22	3,800 3,860	3,830	6,660 6,780	6,720
2	15	30	3,860 3,700	3,780	6,850 6,900	6,875
2	36	40	3,550 3,580	3,565	6,280 6,540	6,410
3	15	51	3,520 3,640	3,580	6,040 5,850	5,945
3	50	58	3,210 3,240	3,225	6,040 5,260	5,650
4	30	69	3,420 3,050	3,235	5,040 6,070	5,555

Sub-base Raft

The sub-base raft was 8 inches thick, and made of 12 : 1 concrete with a water/cement ratio of approximately 1·0. In the early stages of construction, when it was found impossible to compact gravel satisfactorily under winter conditions, dry 12 : 1 concrete was used as filling underneath this raft and gave very good results when rolled with smooth-wheel rollers. It was never found possible, however, to use this dry concrete to the level of the underside of the 12-inch top slab because a sufficiently true finished level could not be obtained. It is possible to obtain a satisfactory level if the aggregate used is flaky and contains a fairly high silt proportion, but with a clean well-rounded aggregate, movement under the roller is too great. The sub-base raft was, therefore, laid with spreading and finishing machines of the same type as used for the top slab.

To prevent congestion, work on the sub-base raft was always kept well ahead of work on the top slab.

12-inch Top Slab

Forms for the top slab were constructed of 8-inch heavy-duty roadforms to the base of which two 3-inch-square hardwood battens were bolted, giving an effective overall depth of 11 inches. The forms were bedded on dry sand-mortar to give a minimum depth between them of $11\frac{1}{2}$ inches and an average of 12 inches. This method was adopted to allow for slight irregularities in the surface of the sub-base; had 12-inch-deep forms been used, the minimum depth would have been slightly more than 12 inches and the average considerably more, resulting in a serious waste of concrete.

The position for the lines of forms was marked out on the surface of the sub-raft and holes drilled, three for each form, with compressors and rock drills using tungsten-carbide bits. Holes were made $\frac{7}{8}$ inch diameter and the forms were fixed rigidly in position by driving in 1-inch-diameter steel pins, 15 inches long.

The operations involved in placing the 20-foot-wide bays of concrete were:—

(1) Preliminary placing of 1-inch-thick expansion joints every 120 feet.
(2) Tipping concrete and spreading with a travelling paddle-type spreading machine.
(3) Vibrating and finishing with a machine operating at 3,000 vibrations per minute. A drive was taken off the engine of this machine to operate two poker vibrators used for ensuring proper compaction along the sides of the forms.
(4) Final surface finish imparted by dragging a heavy canvas belt across the surface as left by the finishing machine.
(5) Forming contraction joints by driving a bar of high-tensile steel, 4 inches deep by $\frac{3}{8}$ inch wide, flush into the surface of the concrete.
(6) Trowelling arrises of joints and sides of bays with a bull-nose trowel.
(7) Spraying surface with bituminous emulsion for initial curing purposes.
(8) Covering concrete with a double layer of hessian (in winter, straw was also used).
(9) Withdrawing bars used for forming joints.

The last operation took place about 90 minutes after the concrete was placed. To obtain maximum outputs, placing of concrete was continued until finishing time and overtime was worked to complete the succeeding operations. Artificial light was necessary in winter.

Bays of non-standard shape at intersections, channels, etc., had to be spread and compacted by hand. For this purpose a beam on which a small petrol-driven vibrating unit was fixed, was used. For bays more

than 12 feet wide, two units were fixed on the beam to ensure uniform vibration.

Some output details are given in Table 3.

TABLE 3

Year	Output: cubic yards			Remarks
	"H" quality	Other qualities	Total	
1945	219,005	184,089	403,094	Four dual-drum, five continuous mixers
1946	175,717	76,875	252,592	Four 2-cu. yd, five continuous
1947	116,745	125,354	242,099	Two dual-drum, four ,,
1948	141,751	97,560	239,311	Three dual-drum, four ,,
1949	130,345	123,878	254,223	Two dual-drum, four ,,
1950	139,103	121,537	260,640	Two dual-drum, four ,,
1951	49,451	49,957	99,400	One dual-drum, two ,,
	972,117	779,250	1,751,367	

The number of mixers indicated in Table 3 cannot be directly related to the outputs shown because there was some overlapping of the number of mixers from year to year, but they give some idea of the mixing strength employed.

The peak day's output of concrete was 3,160 cubic yards, the peak week's 15,746 cubic yards, and the peak month's 57,896 cubic yards, all achieved during the summer of 1945 when an 11-hour day was being worked.

DUCTS AND STORMWATER DRAINAGE

For the airfield lighting system a total of approximately 156 miles of two-way and four-way cluster, 4-inch-diameter, earthenware conduits were laid. All these ducts occurred beneath the concrete paving, and the lighting pits, of which there were altogether 8,700, were formed in the paving itself. Because of the intricacy of the ducting system and the very large quantities involved, laying had to be carefully programmed to avoid hold-ups to the bulk concreting work. The operations involved did not lend themselves so readily to mechanization as other phases of construction and a large proportion of the total labour strength was always kept occupied with this work.

The invert of the four-way duct was 2 feet 3 inches below the top surface of the concrete paving, or 7 inches below the bottom of the lean concrete sub-raft. The top of the duct was, therefore, above the level of the bottom of the sub-raft, and the top of the 6-inch concrete surround very nearly coincided with its surface. The method of construction adopted was to dig the shallow trench necessary, immediately after compaction

of the gravel sub-base had been completed. The "E" quality concrete bed was then placed and the ducts laid on it before any initial set had taken place, so that they were fully bedded. Metal forms 8 inches deep were then pinned in position on each side of the duct trench, set carefully to the level of the top of the concrete sub-raft, and the space between them was filled with "E" quality concrete. After removal of the forms, expansion jointing was laid along each side of the concrete beam thus formed and the bays of the sub-raft were laid up to it. The lighting pits up to sub-raft level were formed by concreting around simple collapsible metal boxes. When the 12-inch top-raft was laid the only problem involved by the ducts lay in forming the upper section of the lighting pit; since this was of quite large dimensions, and also involved forming a rebate for the cover and recesses for the lighting unit, quite a complicated metal box was needed. This is shown in Figs 6, Plate 2.

Considerable use was made of metal boxes of various kinds in forming lighting pits, surface-water catchpits, etc., in the runway surface, and they were uniformly successful. The initial cost was high but many hundreds of uses were secured from them for very little maintenance, and they proved to be most economical.

No special problems were encountered in laying the surface-water drainage. For the most part, of course, this consisted of very-large-diameter concrete tubes laid to very small falls. To ensure proper laying to these falls a concrete raft was laid to accurate levels in the bottom of the trench and the tubes placed on it. Even then some trouble was experienced because of slight variations in the thickness of the tubes, which could cause an apparent backfall; care had, therefore, to be taken to match the tubes before laying and to keep apart tubes from different manufacturers.

Except in the main outfalls and in the early stages of construction, not much trouble was experienced from water because the pumping-out of gravel borrow-pits in various parts of the site had lowered the general water table below the drainage inverts. For the same reason very little timbering was required anywhere.

The main outfalls were substantial projects in themselves, entailing as they did up to four 54-inch-diameter tubes, in places 15 feet deep. Because of the cost and scarcity of timber the trenches were dug with battered sides; water was a major problem and constant pumping from deep sumps formed at the manholes was necessary to keep the water table down sufficiently.

The Site Laboratory

In addition to the control exercised by the employing authorities from their main laboratory, the contractors also operated their own site laboratory. Moreover, testing engineers were appointed to each major concreting plant.

The principal duties of this laboratory organization were :—

(1) Keeping complete weather records.
(2) Testing samples of aggregates, etc., prior to obtaining the employing authority's approval.
(3) Making test cubes. Up to December, 1951, the total number of test cubes made on "H" quality concrete was 32,218, of which approximately one-half were cast by the employing authority and half by the contractor. It is interesting to note that this represents one cube to every 28 cubic yards of concrete. The statistics of the Heathrow test cubes have already been fully dealt with [4] but Table 4 (see Appendix) has been included because it takes the results obtained up to a later date. Columns giving the water/cement ratio have been included because they may be of interest; they include the average of all tests taken on the moisture of the combined aggregates and the average of the amount of water added at the mixer, and whilst it is obviously impossible that any individual test would be correct to three decimal places, the number of tests (from which the figures given have been calculated as averages) is so large that these figures are probably reasonably correct as they stand. The variations occur because, as has been explained previously,[4] the theoretical figure for the quantity of water to be added was used only as a guide and was, in fact, modified slightly to maintain a uniform workability as judged by eye at the wet hopper. There is no doubt that had the theoretical quantity of added water been adhered to, larger variations in workability would have occurred than with the empirical control exercised, but it seems not unlikely that, despite very careful control of the aggregate grading, these variations depended upon factors other than the water/cement ratio; and the departures from the theoretical added water quantities did, in fact, represent departures from the total water/cement ratio which compensated for other factors present in maintaining a reasonably uniform workability. This conclusion is suggested by the close relationship between the theoretical actual water/cement ratio given and the 28-day test-cube strengths.
(4) Control of grading and moisture contents of all aggregate deliveries. Grading tests were taken to ensure that aggregates complied with the specification, but the moisture contents at the time of delivery did not affect concrete mixing because aggregates had to stand for draining for 24 hours after delivery, when the moisture content for mixing purposes was

[4] See reference 1, p. 2.

determined, and the first set of moisture contents were taken solely for commercial purposes. All aggregates were purchased by weight and came mostly from only a comparatively short distance from the site; thus if they were delivered straight from the washing plant without intermediate stockpiling, or during wet weather, the moisture contents at time of delivery became unduly high and unless controlled would have resulted in the loss of very large tonnages of material. What were considered to be fair and reasonable moisture contents were, therefore, agreed with the various suppliers and the actual figures, found by test, were averaged monthly; these results were then given to the suppliers with an intimation of the percentage weights which would be deducted from their deliveries during the coming month as the result of any excess above the agreed figures. This system worked very smoothly in practice and, since the suppliers had a standing invitation to be present to check any tests, very few disputes arose. As a matter of interest, the agreed figures, together with the actual average results, are given in Table 5 (see Appendix). It will be seen that, in all, nearly 40,000 of these moisture tests were taken. As was to be expected, the sand was the only material in which large variations occurred. In Table 6 of the Appendix, average moisture contents of sand immediately before use, that is, 24 hours after delivery, are shown, compared with the as-delivered figures, to show the effects of the drainage period in reducing and making uniform the moisture contents.

(5) General technical control of concrete-mixing set-ups, for example, weekly checking and calibration of the scales on the weigh-batching plants, calibration of continuous mixers, etc.

(6) General control of the testing for density of compacted gravel fill. Owing to the paramount importance of speed in producing these results, this work was decentralized to a large extent and an engineer was placed full time on each section of work for carrying out density tests. The total number of tests carried out was more than 30,000.

In addition to performing the routine duties listed above, the site laboratory carried out any special testing and investigations required in conjunction with the contractor's main laboratory.

Site Organization

Table 7 shows a typical staff organization for work of the magnitude which has been described. Central control was exercised by a resident

manager with a senior agent as deputy. Working directly under these from a central office were the following :—

(1) A chief quantity surveyor responsible for measuring up the work, agreement of day-works, rise-and-fall accounts for materials and wages, etc. The chief quantity surveyor controlled a large staff of quantity surveyors, assistants, and clerks.

(2) A chief engineer who was responsible for agreement of site levels and earthwork quantities, major setting-out and establishment of bench-marks, and for the allocation of engineers to the various sections of the work. The chief engineer worked in close liaison with the quantity surveyors and with the sectional chief engineers.

(3) A progress and planning engineer responsible for programming operations, comparing performance with programmes, planning earth-movement operations, working out plant and transport requirements, designing and positioning major installations, etc. On this side of his duties the progress engineer worked in close conjunction with the resident manager in developing broad plans into detail and keeping him advised of progress trends and tendencies. He was also responsible for the operation of the site laboratory and the control of materials testing engineers and field density engineers. Under him worked a chief laboratory engineer with his staff, a design and records engineer, materials testing engineers, etc.

(4) An office manager in charge of the routine office administration of the contract.

The actual supervision of operations was split up into sectional areas, with a section agent in charge of each one. Generally, at Heathrow, the aim was to decentralize supervision so far as possible and to this end each section was organized to have as much independence as practicable, consistent with adequate regard to the maintenance of a balanced programme of operations and the co-ordination of the utilization of plant, transport, materials, and labour. This decentralization was made particularly necessary by the very large area of the site and the volume of work going on at any one time; in practice it worked extremely well. Co-ordination was secured by the day-to-day control of the central organization and the device of weekly meetings of all heads of sections and departments under the chairmanship of the resident manager. To facilitate liaison with the employing authority, weekly meetings were also held between the senior members of the chief resident engineer's staff and of the contractor's staff under the chairmanship of the chief resident engineer. Control by the employing authority's staff was departmentalized by operations rather than by areas as in the case of the contractors, and hence there was an overlap between the two staffs; whilst this complicated liaison slightly it ensured a uniformity of procedure over the whole site.

TABLE 7.—SITE ORGANIZATION

- Resident Manager
  - Senior Agent
    - Progress and Planning Engineer
      - Laboratory Engineer
      - Design and Records Engineer
      - Section Quality Engineers
      - Materials Testing Assistants
      - Density Testing Assistants
    - Chief Quantity Surveyor
      - Quantity Surveyor Clerks
      - Daywork
      - Rise and Fall
      - Materials
    - Plant Manager
      - Office Staff
      - Foreman Fitter
      - Fitters
      - Storemen
    - Welfare Officer
      - Office Staff
      - Camp Assistant
      - Nursing Staff
    - Chief Engineer
    - Section Agents
      - Section Engineers
      - Engineers
      - Section Foreman
      - Working Gangers
    - Transport Manager
      - Office Staff
      - Foreman Fitter
      - Fitters
      - Storemen
    - Trades Foreman
      - Joiner
      - Plumber
      - Bricklayer
        - Section Charge-hands
        - Tradesmen
    - Office Manager
      - Cashier
      - Wages Clerks
      - Income Tax
      - Timekeepers
      - Secretary
      - Typists
      - Comptometer Operator
      - Telephone Operator
      - Labour Manager
      - Cost and Bonus Clerks
      - Order Clerks
      - Materials Clerks
      - Checkers
      - Storekeepers

Trade Union Federation Officer
Stewards

Catering Manager

Site Inspector

In addition to the weekly meetings referred to, fortnightly meetings of the Production and Welfare Committee, described below, were also necessary and a possible criticism of the system adopted is that so much of the time of the contractor's senior staff had to be spent in attending routine meetings. This was particularly so during the first year or two of the contract when many new problems had to be thrashed out. In practice, however, much of the time spent at these meetings was after working hours and the co-ordination secured was invaluable.

Section agents had working under them :—

(1) A general foreman responsible for the control of plant, transport, and labour. Working under him each section had its complement of assistant foremen, plant and labour gangers, and transport slashers.

(2) A sectional chief engineer working in close liaison with the chief engineer and chief quantity surveyor. Under him worked numbers of engineers and assistant engineers responsible for setting-out, interim measurements, keeping records, etc.

In addition, each section had its own timekeepers, plant checkers, and other clerical staff working under an assistant office manager. Wages and cash were, of course, handled centrally.

Separate and independent organizations were also established for controlling plant and transport. The managers of these organizations were responsible to the resident manager for the efficient maintenance of the machines under their control and for providing experienced operators. Repairs and maintenance were carried out up to the major overhaul stage ; to handle the numbers of machines involved this necessitated elaborate repair-shop installations and a large staff of fitters. Some idea of the scale required may be gained from the following details of the number of machines employed at peak :—

Transport Organization.—261 lorries, 30 vans and private cars.

Plant Organization.—47 mechanical excavators of ½-cubic-yard to 1½-cubic-yard bucket-capacity, 37 tractors and bulldozers, 32 scrapers, 10 large concrete-mixers, 11 elevating graders, 6 concrete-spreaders, 7 concrete-finishers, 29 sheepsfoot and rubber-tired rollers, 16 diesel and steam rollers, 70 concrete-vibrators of various types, 50 water pumps of 2-inch to 12-inch diameter.

Bulk petrol storage tanks were installed at the transport depot and bulk diesel oil storage at the plant depot. Fuelling of plant was carried out by 1,000-gallon tanker wagons with storage compartments for both petrol and diesel.

Ordinary field maintenance of plant was carried out after working hours and on Saturday afternoons. Lorries were serviced by rota on a night shift. A fairly large range of stores was carried but the proximity

of the work to London made this problem less formidable than it might have been.

Requirements and allocation of plant and transport to the various sections was programmed in advance and adjusted as necessary at the site meetings. After the plant and transport organizations had met requirements, the section agents were responsible for the efficient operation, as distinct from maintenance, of the machines allocated to their section.

Also controlled centrally were the tradesmen, that is, joiners, bricklayers, and plumbers. Each trade had a foreman in charge operating over the whole site; they maintained close liaison with the section agents and supplied tradesmen and charge-hands as required. The joiner foreman was also responsible for the central joiners' shop where timber and hutting was stocked and also for the fabrication of all joinery carried out.

Welfare and catering were important aspects of the site organization. The number of men employed ranged from 1,500 to 1,900 and, since most of these came from outside the London area, arrangements had to be made to house a large number on site. From 1944 to 1948, men were housed in a temporary camp of timber hutting, but later on, when it became apparent that the need for this type of accommodation would be a long-term one, a permanent camp to house 1,000 men was built. This camp included barrack blocks, canteen, wet canteen, recreation hall and cinema, sick bays, staff quarters, etc. The whole installation was centrally heated and it was a model of its kind. The building of this camp alone was quite a major project.

Catering was supervised by a catering manager responsible for the whole site. In addition to the camp canteen, which accommodated 1,000 men, three other canteens, each serving 500 men, were operating on site for serving mid-day meals. These site canteens were of timber construction and had to be dismantled and re-erected as the centre of gravity of operations changed. Mobile canteens were used for serving men working in outlying areas.

Great importance was always attached to the maintenance of good relations with the workmen. This was secured by setting up a Production and Welfare Committee and by the employment of a full-time Trades Union Federation steward. The Production and Welfare Committee was composed of members of the contractor's staff and of elected representatives of the workmen, meeting fortnightly under the chairmanship of the resident manager. Joint secretaries were appointed, who agreed on an agenda beforehand. At these meetings any complaints affecting welfare were discussed and remedies agreed; also, ideas for securing better production were sought after. The Federation steward was responsible for all matters affecting trades union organization on site; he also dealt with the management on all routine queries which might arise affecting such things as pay, bonus, allowances, etc. Any serious difficulties or disagreements which might arise were discussed between the resident

manager and the Federation steward and were usually quickly resolved. In general, labour relations on the site were excellent throughout and the amount of time lost through disputes was negligible.

The Paper is accompanied by ten sheets of drawings, from which folding Plates 1 and 2 and the Figures in the text have been prepared, and by the following Appendix.

APPENDIX

TABLE 4.—SUMMARY FOR QUALITY "H" CONCRETE (YEARLY AVERAGES)

Year	Concrete output: cubic yards	Cube strength: lb. per square inch		Coefficient of variation at 28 days: per cent	Increase on 7-day strength per cent	Water/cement ratio			Number of cubes tested, including A.M.
		28 days	7 days			Water added by A.M.F.O.T.	M.C. of combined aggregate	Actual water/cement ratio	
1944	3,000 (approx.)	6,641	4,599	—	—	—	—	—	458
1945	219,005	5,844	4,045	9·03	45·0	0·293	0·182	0·475	5,189
1946	175,717	5,642	3,731	8·64	51·3	0·304	0·183	0·487	4,704
1947	116,745	5,690	3,770	9·08	51·1	0·300	0·178	0·478	4,350
1948	141,751	6,123	4,238	8·06	45·3	0·301	0·176	0·477	5,964
1949	130,345	6,182	4,272	7·96	45·3	0·305	0·174	0·479	4,400
1950	139,103	6,435	4,548	5·96	42·2	0·287	0·182	0·469	4,115
1951	49,451	6,018	4,081	7·84	47·5	0·296	0·176	0·472	3,038
	972,117	5,991	4,098	8·08	46·8	0·298	0·179	0·477	32,218

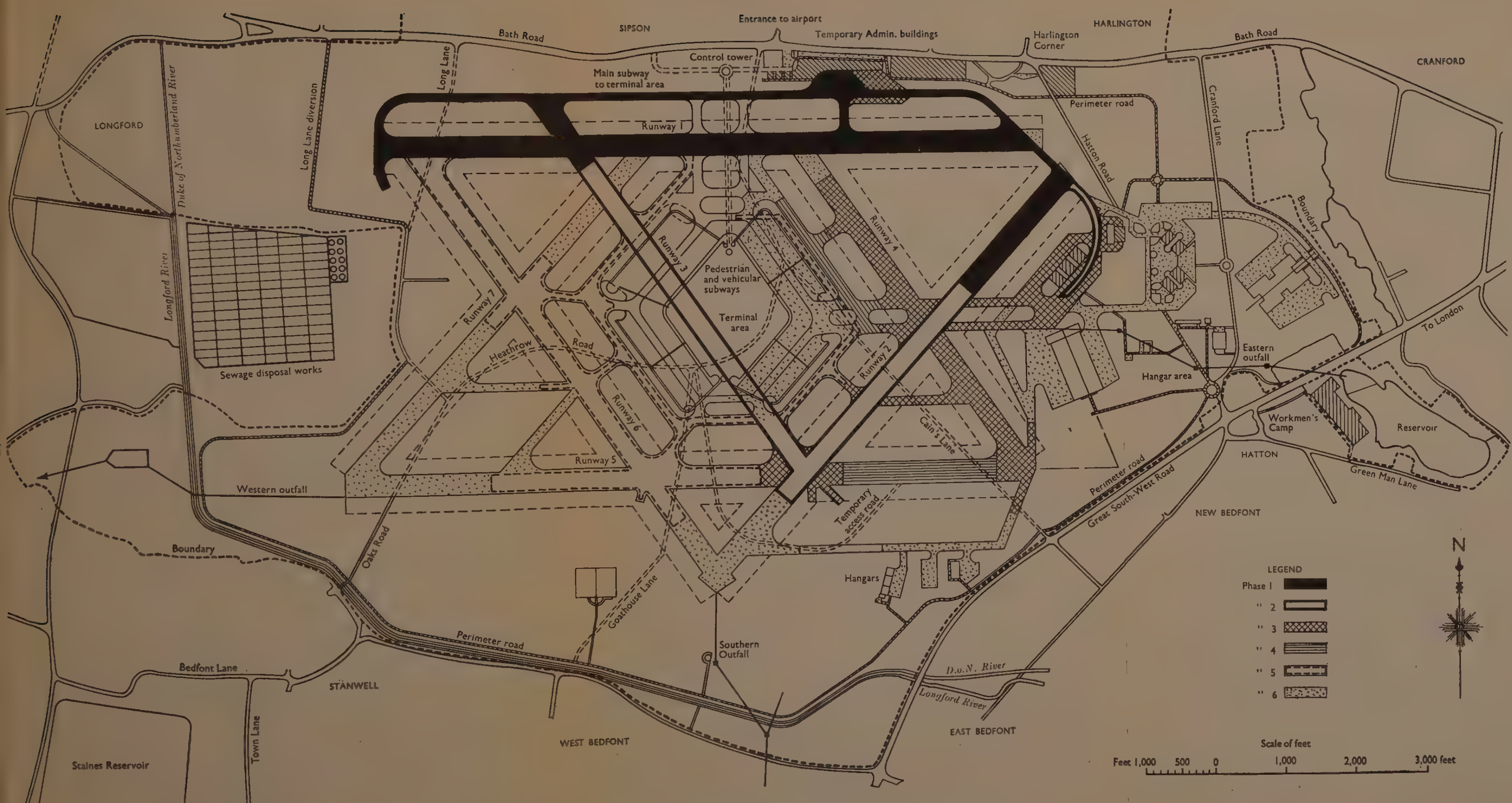
TABLE 5.—SUMMARY OF ABSOLUTE MOISTURE CONTENT OF ALL AGGREGATE AT TIME OF DELIVERY

Year	Number of tests ("H" aggregate)	$1\frac{1}{2}''$–$\frac{3}{4}''$	$\frac{3}{4}''$–$\frac{3}{8}''$	$\frac{3}{8}''$–$\frac{3}{16}''$	Sand	$1\frac{1}{2}''$ all-in ballast	$\frac{3}{4}''$ all-in ballast	Number of tests on all-in ballast
1945–46	7,665	1·6	2·9	4·2	9·0	6·5	7·1	681
1947	5,028	1·1	2·4	4·4	9·2	6·8	5·9	1,709
1948	6,006	1·3	2·6	4·9	11·0	5·2	5·1	2,259
1949	5,055	1·3	2·5	4·2	11·0	7·2	6·1	1,952
1950	4,651	1·2	2·2	4·5	11·9	6·5	6·5	1,520
1951	2,093	1·2	2·1	4·5	10·7	5·5	7·9	1,023
	30,498	1·3	2·5	4·4	10·5	6·3	6·4	9,143
Agreed reasonable figure		1·0	3·0	5·0	7·6	5·0	6·0	

TABLE 6.—SURFACE MOISTURE CONTENTS OF SAND BEFORE AND AFTER 24 HOURS' DRAINING

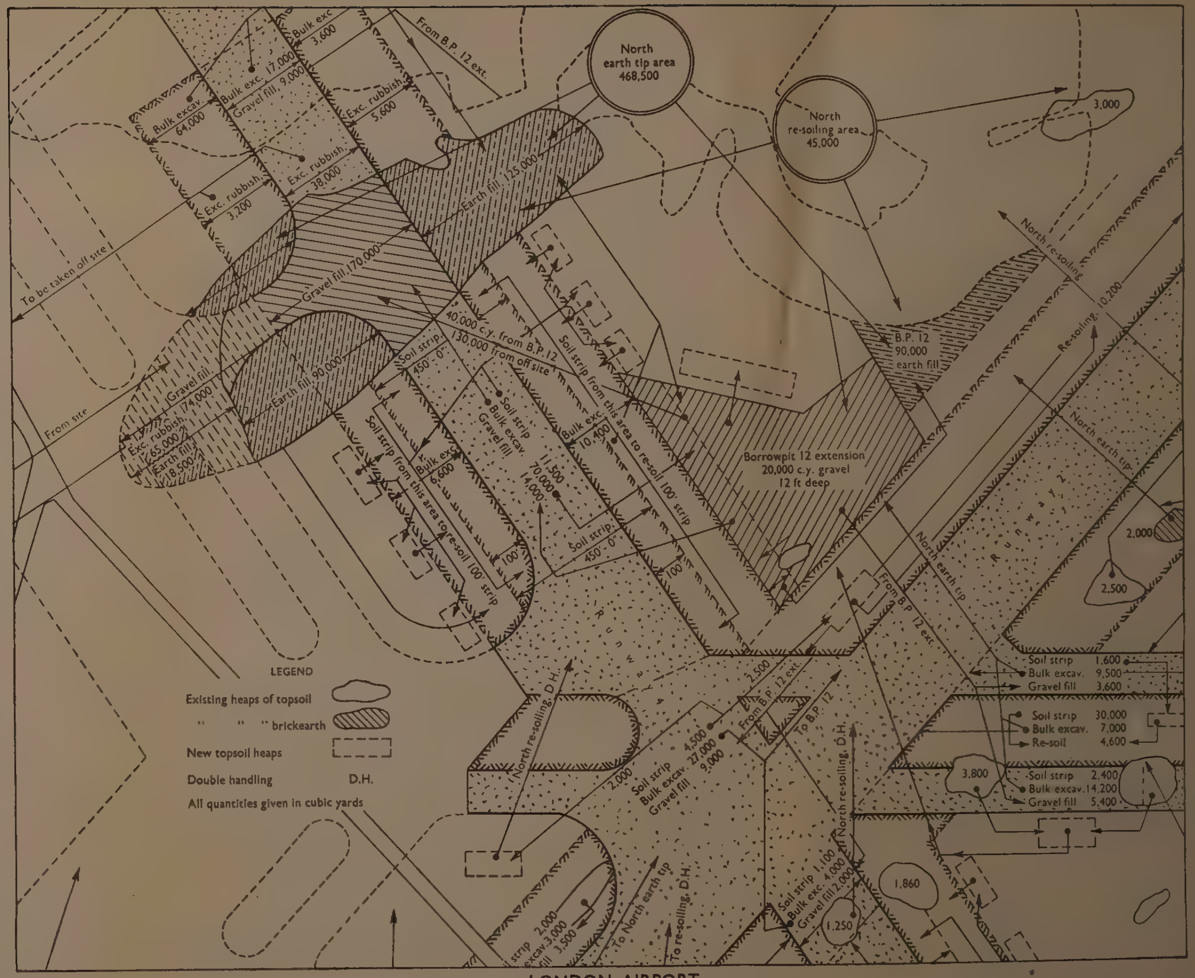
Year	Average moisture content at delivery	Average moisture content after draining
1945–46	8·2	5·9
1947	8·4	5·7
1948	10·2	5·9
1949	10·2	5·8
1950	11·1	5·9
1951	9·9	5·6

FIG. 2



LONDON AIRPORT
SITE PLAN SHOWING PHASING OF WORK AND SITE ACCESS ROADS

FIG. 3



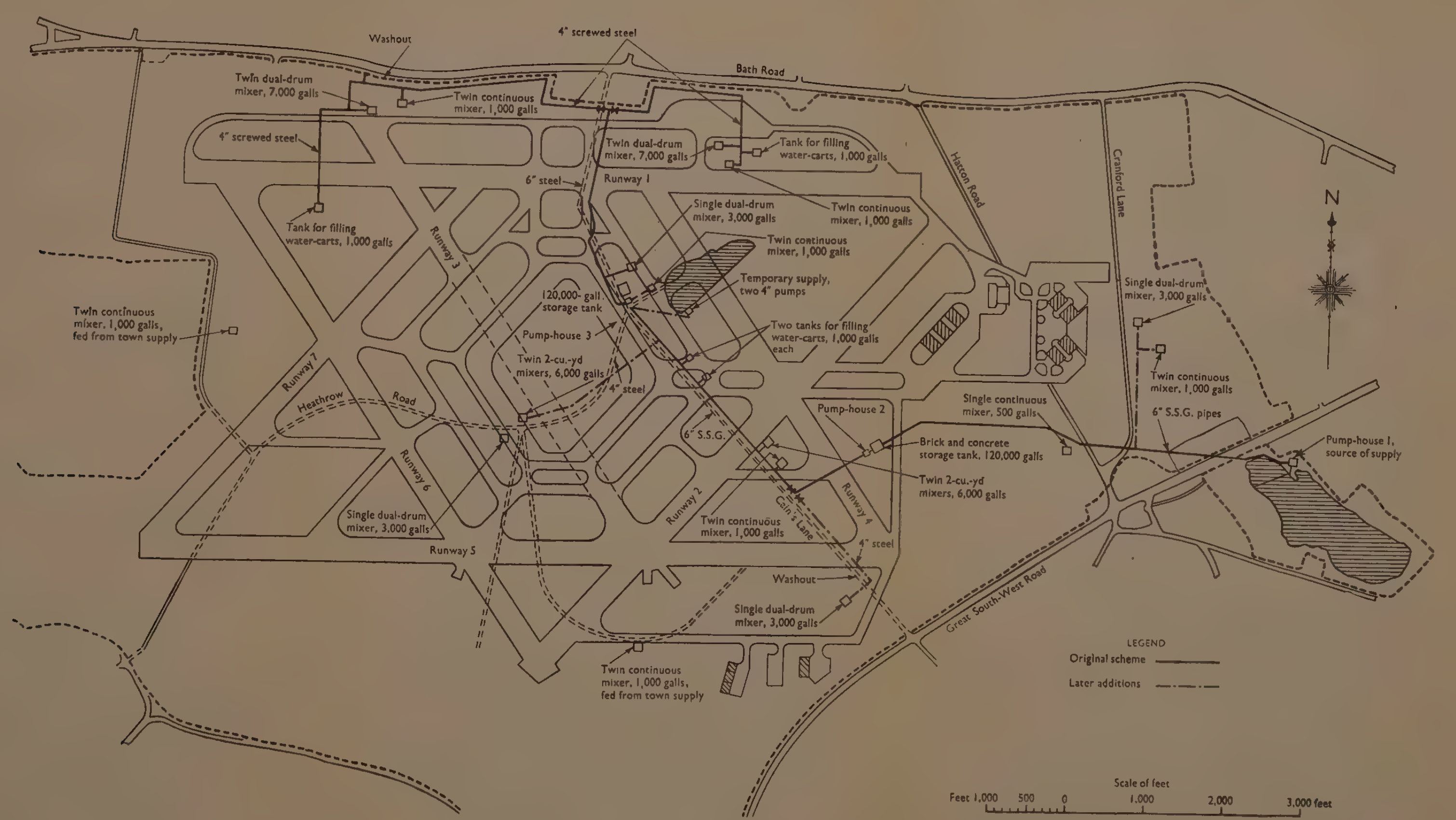
LONDON AIRPORT
TYPICAL SECTION OF EARTH MOVEMENT PLAN

E. A. PALMER

...LLIAM CLOWES & SONS, LIMITED: LONDON

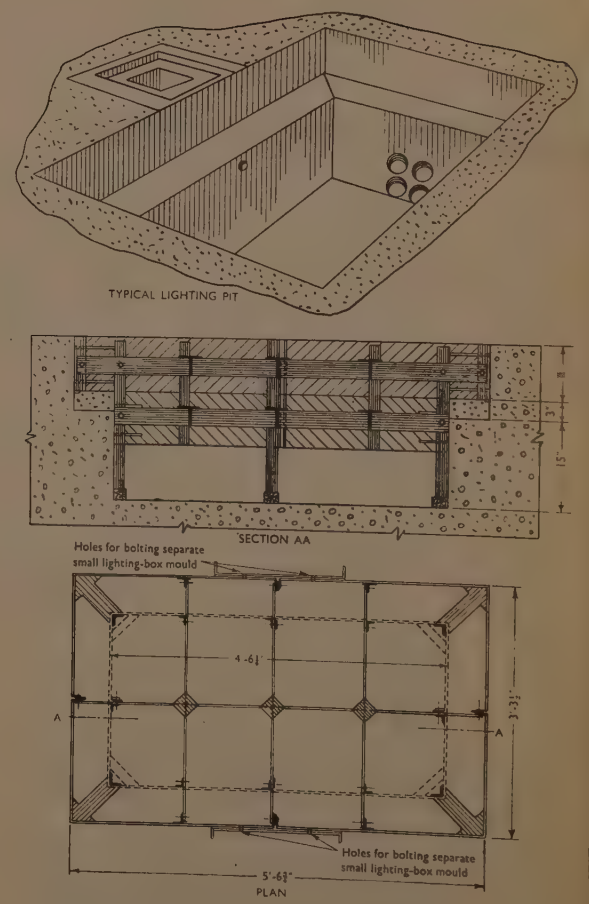
THE ORGANIZATION OF AIRPORT CONSTRUCTION

Fig. 5



LONDON AIRPORT
LAY-OUT OF TEMPORARY WATER SUPPLY

Figs 6



COLLAPSIBLE STEEL MOULD FOR FORMING LIGHTING PITS IN 12-INCH TOP SLAB

E. A. PALMER

William Clowes & Sons, Limited: London

Discussion

Mr J. A. Dawson observed that in the Introduction to the Paper, the Author had stated that "it would be difficult to generalize on the subject of the organization of airport construction." Mr Dawson did not think that it was necessarily impossible to do so in respect of the majority of airports other than London Airport, but the constructional problems which had had to be solved in the execution of the London Airport project, which was by far the largest of its kind ever undertaken in Great Britain, had differed not only in scale but also in character from those met elsewhere. Different sites had presented their own special problems, arising out of the nature of the subsoil, the location of watercourses capable of receiving a high rate of surface discharge, and the availability of materials required for the construction of the runways and other paved areas; but most other airfields constructed in Great Britain, including those provided during the 1939–45 War, had had two common characteristics: the complete runway system to the initial dimensions had been constructed before the airport was brought into use, and the plan had been based on a graded scheme of cut-and-fill, balanced within the site. Neither of those conditions had obtained in the case of London Airport.

As shown in Fig. 2, Plate 1, of the Paper, the work at London Airport had proceeded in six phases, and from an examination of the areas covered by those phases it would be noted that they had not followed what might be regarded as a suitable sequence from the constructional point of view. They had conformed, as they inevitably had to conform, to flying operational requirements, and to some extent also to the timing of the acquisition of the land.

The airfield initially planned for the use of the Royal Air Force comprised Runways 1, 2, and 3 (shown in solid black or edged with thick black lines in Fig. 2, Plate 1); the remainder had been embodied in the plan prepared at a later date for the civil airport. The programme had not permitted the construction of even the first three runways before the airfield was brought into use. The main east-west runway, No. 1, had had to be completed first and, although the airport had been taken over for civil purposes before flying commenced, No. 1 Runway had still had to be used before the other two were available. Thereafter, the planning of the work had been greatly influenced by the fact that the airport had been in continuous use after No. 1 Runway had been brought into use.

The complication of the phasing necessarily affected the organization of the constructional work, and probably the Author would agree that the siting of the concrete-mixing installations, for instance, would have been very different had it been possible to proceed in accordance with a master plan based on engineering considerations only. On the other hand, as the Author had agreed, the actual programme had permitted continual

improvement in method, and Mr Dawson thought that that was particularly noticeable in connexion with the lay-out of the concrete-mixing plant. Dispersal in the first set-up had given way to concentration, so that vehicular transport within the mixing set-up—that was to say, at any point between receipt and issue points—was eliminated, whilst the adoption of an elevating boom in Type 4 had reduced the height of the whole installation. Those changes had inevitably resulted in greater efficiency and economy.

As was stated on p. 2 of the Paper, the site of the airport was flat agricultural land, and that might be emphasized by stating that the fall from one side of the site to the other was no more than 5 feet. A cut-and-fill scheme in the ordinary sense had therefore been unnecessary, but the removal of the brickearth, and its replacement (over the natural gravel) by gravel excavated from borrow-pits on the site had been in effect a cut-and-fill scheme of unprecedented size and character. It was to the credit of the methods adopted to consolidate the gravel that, as noted on p. 10 of the Paper, there had been no settlement in the re-filled gravel pits, just as there had been none in the filling elsewhere. Under heavy traffic, the concrete paving (566 acres) had maintained its original level.

On p. 4 the Author had stated that the imported gravel had been obtained from pits outside the safety lanes and safety margins of the paved surfaces; to that should be added, said Mr Dawson, as the Author would no doubt agree, the areas allocated for building or which might be required in future for building purposes. The Author would probably also agree that the minimum dry density of 126 lb. per cubic foot mentioned on p. 8 should be 128 lb., which was the figure in the specification. Mr Dawson hoped that it would not be inferred that the contractors had been giving the employing authority 126 lb. when the latter thought that they were getting 128 lb., because that would be quite untrue. Actually the minimum of 128 lb. per cubic foot had been rigidly adhered to, but on the average the figure ran very much higher, and was probably about 135 lb.

The construction of the airport had obviously necessitated the accurate setting-out of the runways and taxiways, and it would have been of interest to have a description of the steps taken to ensure the accuracy of their orientation and position. Mr Dawson mentioned that the brass cannon marking the western end of General Roy's base-line for the first trigonometrical survey of the country stood about 60 feet north of the taxiway to No. 1 Runway, and had been used as the fixed point from which the runway and taxiway were set out.

Would the Author enlarge on the question of plant repairs and the need to ensure that plant was maintained in a satisfactory condition, to which slight reference was made on p. 26 of the Paper? It was of the utmost importance to contractors that plant should continuously be fit for work at maximum output and that labour was not unemployed through failure of the wheels to turn. That was also of great importance to the

employing authority, who wanted breakdowns reduced to the absolute minimum, to ensure the planned rate of progress.

Finally, Mr Dawson wished to emphasize the importance of good relations with the labour employed, since good relations led to high output, beneficial to all concerned. It was interesting to note that great importance had been attached to that matter, and that the amount of time lost through disputes had been negligible; there had also been a very good team spirit amongst a considerable section of the men.

Mr Harold Smith remarked that Mr Dawson had covered a good deal of the ground which he himself had intended to traverse, but there were one or two items about which he would like to have further information. Table 3 gave the outputs of the concreting machines, and Table 1 gave the efficiency, presumably based on working hours. The overall outputs were something like half that figure, which seemed to indicate that the centres of concreting had not been in the best positions, a point to which Mr Dawson had referred.

On p. 22 the Author had referred to test cubes and had given very remarkable figures of the test-cube results on the job, also mentioning that more than 32,000 test cubes had been made. Mr Smith wondered if all that number had been necessary. The Author had stated that the water/cement ratio had been based on the total amounts of aggregate, water, and cement used, and taking the overall figures he had assessed the water/cement ratio as 0·477. That was no doubt perfectly true, but it would be interesting to know the deviation from that in one day's working. Another point was that the aggregate/cement ratio played a very important part in those figures. It was not possible to take the water/cement ratio alone, because if the aggregate/cement ratio was varied, it was not necessarily the concrete which the contractor had tendered to supply. It would be interesting if the Author could give some indication of what the aggregate/cement ratio had eventually been, and what that meant in terms of tons of aggregate, sand, and so on.

The method which Mr Smith liked to use, and one which he had advocated before and tried out on site, was to take densities of the "plastic" concrete and obtain the calculated mean density, and from that find a standard deviation over a dozen or so tests carried out during the morning for a definite quantity of concrete which was going to be cast during the day. It had been found that it was possible to get very accurate results with regard to the yield, which was most important to the contractor and also to the employing authority. It was possible to see immediately the trend of the concrete which was being used and its consistency, which would tell beforehand the type of result which would be obtained from the test cubes. It was necessary to wait 7 or 28 days for test-cube results, but by knowing the density on the job each day the strength or the quality of the concrete could be determined. On one job at London Airport with which Mr Smith had been concerned, that method had been tried; in one case it had been

estimated from the calculated density that 350 cubic yards would be required. Tests of density had been taken during the morning—not making any allowance for compaction—and it had been calculated from the deviation obtained over the few tests made that 351 cubic yards of concrete would be required. Actually the figures had varied between 352 and 349 cubic yards. If that was maintained over a long period of time it would give a very accurate result, and make it possible accurately to forecast the requirements of cement and aggregate for any big job, which was an important factor to-day.

Mr G. S. Cooper said that, in Table 4, the Author had given coefficients of variation, and for 1950 there was the very striking figure of 5·96 per cent, which was extremely low and pointed to very good control. Graham and Martin[1] had analysed the position in 1945, and had arrived at a figure of 9·8 per cent; they accounted for half of that variation by variation in the strength of the cement itself. The figure given by the present Author would mean that the coefficient of variation was 50 per cent less than when Graham and Martin had written their Paper. If the variation of the cement had remained constant, then all other causes of variation would have had to be eliminated. It would be interesting to know if the Author had any evidence that the consistency of the cement had improved, so as to account for the difference, or could say what other factors had influenced the very considerable reduction in the coefficient of variation.

On p. 18 of the Paper, reference was made to the sub-base raft, and it was mentioned that there had been some difficulty in obtaining a sufficiently true finished level of the lean-mix concrete by rolling, and that resort had had to be made to vibration. Presumably that had entailed an increase in the water/cement ratio to get the machines on to it, and it would be interesting to have some further information about that, because it was not clear why the compaction could not have been obtained by rolling. It was stated that it was possible to obtain satisfactory compaction by rolling if the aggregate used was flaky and contained a fairly high silt proportion, but that seemed to be only a question of aggregate grading —which could be achieved without any special effort.

Mr Alan P. Lambert asked for further information on two points connected with the preparation of the formation. He knew very little about Proctor densities, but when working on a large gravel site the noticeable feature was not the uniformity of the material, but the way in which it varied, possibly from almost pure sand in one place to gravel with very little sand in it in other parts not far away. He would be interested to know, therefore, how a representative sample was taken from which the Proctor density was obtained, and what degree of variation was in fact experienced in the formation when it was tested for density.

On p. 8 of the Paper the Author had stated that " To achieve the

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[1] See reference 1, p. 2.

required result, the grading had to be such that the quantity of material passing the No. 100 sieve was not less than 15 per cent and not more than 30 per cent of the quantity of material passing the $\frac{3}{16}$-inch sieve," and that "Generally the gravel excavated from site borrow-pits complied with the grading requirements." That seemed very decent of it! In the first paragraph on p. 9, the Author had said that "The optimum moisture content of the gravel for satisfactory compaction was between 4 and 6 per cent, and it was generally found that this was the content which it retained naturally after the borrow-pit had been dewatered." Again Mr Lambert felt that that was very decent of it, but he would like to know what happened when the gravel was not so accommodating! It would appear in that case that it had not been possible to reach the density of 128 lb. per cubic foot required, and the interesting point arising was to discover what had been the tolerance actually allowed in practice against the theoretical figure of 128 lb. per cubic foot.

Mr Lambert then referred to the degree of accuracy possible when preparing a formation to a certain level. His experience had been that it was difficult when using mechanical plant to work to a fine degree of accuracy, and he would consider a tolerance of only ± 1 inch to be a very high degree of accuracy. It was stated in the Paper that a great deal of the material had been loaded by draglines into lorries, tipped into place, spread by bulldozers, and then compacted by sheepsfoot and rubber-tired rollers. Was it to be understood that that process left the formation to within ± 1 inch of the required level? He would have thought that an extra process involving a good deal of grading with auto-patrols or scrapers was necessary in order to leave the formation at that degree of accuracy. Would the Author say whether that had also been a difficult problem at Heathrow?

Mr N. N. B. Ordman asked whether the contractor and the employing authority had maintained separate control organizations for the control of the concrete, or whether they had worked together. He also asked if either of them had, to the Author's knowledge, developed any special methods of testing—for example, aggregate moisture testing—which had proved rapid, practical, and accurate, and which were not referred to in the Paper. It would appear that an enormous number of tests had been carried out, and if the normal method of weighing before and after heating had been used, that had surely taken up a great deal of time. Had other methods been employed, and, if so, could something be said about them?

He assumed that the aggregate delivered to the site had been graded at the pits to within certain limits, which had been checked on the site at intervals. Had it been found that the aggregate suppliers had kept within reasonably close limits of the required range?

Lieut-Col. W. M. Glasgow (United States Army), referring to the statement in the Paper that it had been found during the winter months, in the compaction of the sub-grade, that it was impossible to control the

moisture content, and that that had finally led to the substitution of weak concrete, asked whether any work had been done on the use of cement as a drying or soil-stabilizing agent and whether any success had been achieved in that direction.

Mr. J. M. Fisher observed that mobile Paver-type mixers had been used as central mixers. Those machines had been developed in America and were very extensively used in that country, but he did not know that they were ever used as central mixing plants. Could the Author say why, in the planning of the job at London Airport, it had been decided to use mobile mixers as fixed mixers at the central mixing plant?

The Chairman said that Table 1 seemed to show an extremely high state of efficiency of the plant, because in the week in question only one item had been broken down for 5 hours. It would be interesting if the Author could give a little overall information with regard to the availability of plant.

Table 2 gave the effect of the transportation of the concrete, and it almost seemed at first glance that concrete could be improved by carrying it about for 2 hours. He did not know whether there was any reason for that, but would like to know if the point had been investigated.

Mr L. A. Dobbs, referring to the high-tensile-steel bar which had been driven into the surface of the concrete to form the contraction joints, asked whether any difficulty had been experienced in getting the steel into the concrete and in keeping it in a straight line for, presumably, 20 feet.

_ **Mr J. G. Colquhoun** said that, as Chief Resident Engineer from the start of construction in 1944 until 1948, he found the Paper a most interesting and factual description of the methods of construction employed and of the many difficulties encountered, particularly in the early pioneering stages. With regard to the relative staff dispositions referred to by the Author, it was his opinion that, so far as the employing authority was concerned, the disposition of the staff by operations, rather than by areas or sections, was the best arrangement in the circumstances, since the specifications and the methods employed were novel on very-large-scale construction in Great Britain. That method avoided misunderstandings in the interpretation of the various specifications, and the temptation—understandable in certain circumstances—to play off one engineer's opinion or interpretation against another's, where the question of any latitude or departure from the strict requirements of the specification might arise. He understood that, with the routine of operations in respect of concrete paving, etc., well and truly established, and with the more varied scope of the work in the opening-up of the permanent hangar areas, central terminal area, and access tunnels, his successor, Mr Dawson, had more or less re-disposed the staff by sections—which no doubt fitted the circumstances better.

He agreed with the Author on the importance of welfare and the

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_ This contribution was submitted in writing.—SEC. I.C.E.

maintenance of good relations with the labour force. It had been Mr Colquhoun's experience on the contractor's side that, with the emancipation of the labourer from the old days of the elephant hut, the hunk of bread and cheese, and the "drumming-up boy," a contractor's agent was about 80 per cent a welfare and social organizer, and 20 per cent a technical director or administrator. The time devoted to that side of the organization at Heathrow was well justified and amply rewarded, since the relations with the labour were excellent. Pride of workmanship was encouraged and a technique developed in runway construction that was unsurpassed.

The Author, in reply, said that Mr Dawson had given some very interesting background information on the job, and had enlarged a good deal on the Paper. In reply to his specific points, the siting of the concrete installation would definitely have been different had it been possible to plan the job as a whole. One of the mistakes made at the beginning had been that the cost of erecting plants of the type required to produce very large quantities of high-quality concrete had been under-estimated. It had been found by experience that it was cheaper to build the plant in a central position and face the necessity for hauling the concrete a comparatively long way rather than to build plants very near the work and have to demolish and re-erect them later.

The areas for digging gravel did not include the areas earmarked for building, and very close control had been maintained in deciding the areas from which gravel could be obtained. He apologized for the mistake in the Paper where a figure of 126 lb. per cubic foot was mentioned. The minimum dry density was 128 lb., and if a figure of 126 had been obtained it would have been rolled again until the higher figure was obtained. The setting out of the runways and taxiways had been a job on its own, every possible precaution having been taken to ensure accuracy. It was regretted that the length of the Paper had not allowed a description of the setting out but that really merited a Paper on its own. The same applied to the question of plant repairs and maintenance.

In reply to Mr Harold Smith, the Author said that he had already stated that the concrete plants were not in the ideal position but that had little to do with the actual working efficiency of the plants, because the amount of transport available to serve the plant was always adjusted to the length of haul. In fact the positioning of the plants, at least in the early stages, would have tended to increase their working efficiency if anything, because too much emphasis had been placed on reducing hauls to the minimum. The figures given in Table 1 were not meant to be taken as indicating the general efficiency of the plant, but were only given as an example of how efficiencies were tabulated: they applied only to two plants over one week. With regard to the water/cement ratio, available data indicated that it ranged from 0·45 to 0·49 with a tendency for the lower values to obtain in the winter months and the higher during hot dry weather. It was the Author's opinion that little reliance could be placed on the figures for individual days

because the method by which they were determined was too inexact and it was only when an average could be obtained over a very large number of results that a reliable figure could be expected. Mr Smith's remarks on taking the densities of the plastic concrete to show the outputs which would be required were very interesting, but there was a factor which militated against the use of that method for runways, namely, the thickness of the slab. If it were possible to guarantee that that would be exactly 12 inches the method would be a good one, but it was impossible to get a slab 12 inches thick; sometimes it was slightly more and sometimes slightly less. Although the percentage error was not great, it tended to spoil the usefulness of Mr Smith's method because it was impossible to be certain of the exact volume of concrete laid.

With regard to the aggregate/cement ratio it was difficult to give an accurate figure, because rather more of both aggregate and cement were used than was theoretically necessary. In the case of aggregate the excess was just less than 7 per cent and of cement just more than 6 per cent: since the percentage excess of cement was slightly less than of aggregate the actual aggregate/cement ratio, obtained from materials used, worked out at slightly above the specified figure, 6·255 : 1 against 6·207 : 1, but the approximation was very close. Whether the former figure was in fact correct would depend on an exact analysis of the reasons for the excess consumption of materials, which it was not possible to obtain. No doubt, however, the principal reasons in order of importance were :—

(1) In order to make sure the slab was of the specified thickness, the tendency was to make it slightly too thick; a 12¼-inch average slab would account for a 2-per-cent excess of materials and an 8¼-inch average slab for a 3⅓-per-cent excess. It was in fact noticed that the excess tended to rise when 8-inch slabs were being laid.

(2) Some wastage took place at starting-up and finishing times: at starting times the first two or three loads usually had to be condemned because the water/cement ratio tended to be too high, and at finishing times only part of a load might be required to finish off a bay and the rest would be wasted or, if possible, tipped as drain surround. Assuming four loads per day lost from each plant the loss would be in the region of 2–3 per cent.

(3) In spite of all the precautions taken it was possible that small losses of aggregate occurred through excess moisture contents; that was to say, material counted as aggregate was in fact water.

Causes (1) and (2) would not affect the aggregate/cement ratio but any error arising from (3) would tend to make the ratio appear higher than the actual one.

Mr Cooper had drawn attention to the low figure given in Table 4 for the coefficient of variation obtained in 1950. Apart from 1951, when only a comparatively small volume of concrete had been laid under less favourable circumstances, there had been a tendency for the coefficient of variation to diminish each year, which might be attributed to improvements in technique, but other than that, no reason could be given for the low figure in 1950. That certainly seemed to point to an improvement in cement consistency. With regard to the question of the dry lean-mix concrete, it was a question not only of aggregate grading but of the aggregate shape as well. Clean hard round aggregate would not stay in place under the roller; a billowing effect was obtained, and it had been impossible to obtain even an approximately smooth formation. That was why the other method had been resorted to. The Author had seen a job where the first method had been successfully employed, but there almost a laminated aggregate with a very high proportion of silt in it had been used. In that case quite a good formation had been obtained, but the aggregate was entirely different from the one commonly used in the London area, and it would have been impossible to convert the London-area aggregate into a similar grading and shape. Of course, the point arose that a clean well-rounded aggregate could be laid with the higher water/cement ratio necessary for mechanical placing, and the resultant concrete would still compare favourably in strength with that obtained from a silty laminated aggregate rolled in place.

With regard to Mr Lambert's comments, samples had been taken very frequently to ensure that the gravel used was uniform. In fact, it had been found to be very uniform indeed, and probably less than 1 per cent had had to be refused. The water content, 4 to 6 per cent, was important, and it was fortunate that that had been the general figure for the pits after they had been de-watered. There were quite large variations in grading between individual strata but excavation with a dragline gave a mixing action which cancelled out those variations. As had been stated, it was absolutely essential that the pits should be thoroughly dried out before excavation took place; if that was not done a lot of the fines were lost and the gravel could not be compacted. Densities obtained were usually well above the specified figure; in the few cases where they were below, it was usually found that the gravel had been allowed to dry out too much before rolling and it was put right by watering and re-rolling. In a few cases the trouble was lack of fines and the gravel was then dug out and replaced with suitable material.

The Author agreed that it was difficult to obtain a tolerance of only ± 1 inch, and in fact it had not been obtained at first, but after the first season's work the labour had become more skilled and very good results had been obtained. It was not quite true that no further work had had to be done after rolling: the method used was to leave the formation slightly high, a boning gang then going over it removing any surplus material with a grader.

In reply to Mr Ordman, the contractors and the employing authorities did have separate organizations for controlling the concrete, but they worked very closely together. No special method of determining moisture content had been improvised. Generally, the material was dried out in an oven and weighed before and after. That was a tedious process, but it simply meant employing enough staff to do it. They had not had much success with the picnometer method at London Airport: they had found that it did not give such accurate results as the oven method.

Grading tests had been taken on representative loads of aggregate as they were delivered and generally it had been found that the suppliers did remarkably well in keeping their material to the limits required: the percentage which had to be rejected had been negligible.

In reply to Colonel Glasgow, they had not tried using cement as a drying agent in wet weather, as a substitute for weak concrete. The Author was curious to know more about that, because it was not a process which had ever been tried out very extensively in Great Britain so far as he was aware. It seemed akin to soil/cement stabilization, which had never been used very successfully on a large scale in Britain. The unpredictable weather made it impossible to rely on obtaining a moisture content which would lie within reasonable limits.

Mr Fisher had referred to the Paver mobile mixers. Those had been used because they were a complete mixer and formed one unit. They could simply be put in place, and it was not necessary to add anything else, apart from the weigh-batchers. As explained in the Paper, ordinary 2-cubic-yard mixers had been used on two plants, but the cost of erecting those plants had been very high compared with the cost of the plant employing mobile Pavers. The biggest advantage of the paver lay in the large lifting hopper. If a fixed hopper had to be used the batching bins and hoppers had to be put much higher up and that was a very important factor with regard to cost of erection. Also, of course, the dual-drum mixer incorporated in the type of paver used gave a much higher output than any other type of mixer of similar dimensions.

Table 1, to which the Chairman had referred, was intended only as an example of the way machine efficiency was recorded and should not be taken as typical of the efficiencies actually achieved. The actual time lost through breakdown was of the order of 2 per cent for concreting plant, 5 per cent for excavators, and 8 per cent for tractors, bulldozers, etc. In the Author's experience those were unusually low figures and reflected the opportunity afforded by the size and length of the contract, to set up a really efficient maintenance and repair organization.

The results given in Table 2 had surprised the Author. Only one test had been carried out and no further investigations had been made. He believed that a similar improvement had been noticed in concrete which had been vibrated for a long period, and no doubt the action of the lorry during transportation achieved a similar effect.

Mr Dobbs had referred to the high-tensile steel which had been used for forming contraction joints. It had been found difficult to drive the steel into the concrete, which was why high-tensile steel had had to be employed; ordinary steel had been tried to begin with, but it had not been at all successful because the steel had become distorted after two or three uses and the resulting joints were crooked. The method of driving the steel in was to have three or four men hit it with 14-lb. hammers; that sounded crude but no more effective method of applying the considerable force needed had been found.

The closing date for Correspondence on the foregoing Paper has now passed without the receipt of any communication.—SEC. I.C.E.

MARITIME AND WATERWAYS ENGINEERING DIVISION MEETING

21 October, 1952

Lieutenant-Colonel R. H. Edwards, Member, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Maritime Paper No. 21

" Some Designs for Flexible Fenders "

by

Donald Hamish Little, B.Sc.(Eng.), A.M.I.C.E.

SYNOPSIS

Open piled jetties or wharfs are the cheapest maritime berths to construct but, being light and rigid, they must be protected from berthing impacts by fendering systems of adequate flexibility. A mass of 1,000 tons moving at a speed of ½ foot per second has an approximate kinetic energy of 50 inch-tons. If it is assumed that half the weight of a vessel is effective when it comes alongside (that is, that first the bow strikes a fender and then the stern swings round and strikes a second blow), then a vessel of 40,000 tons coming alongside at a speed of ½ foot per second will require a fender of 1,000 inch-tons capacity to receive it safely. Similarly a vessel of 1,000 tons will require 25 inch-tons.

Fendering systems of this wide range are now becoming common practice so that open piled jetties can be built to cater for the largest of ships and in comparatively exposed waters, and a detailed description is given of twenty representative designs selected from work done over the past 10 years.

Post-war developments in the use of rubber in shear (by bonding the rubber to metal plates) and swinging clumps of concrete (essentially a gravity device) are described in some detail.

All experience on all types of fendering—of large and small capacities—leads to the conclusion that longitudinal or glancing blows are as important as those at right angles to the jetty line. They are, however, most difficult to cater for and future development work will be mostly concerned with this particular aspect of the fendering problem.

INTRODUCTION

GRAVITY masonry or concrete walls for wharfs or jetties are undoubtedly the most economical types of structure, from the point of view of maintenance. This is mainly because the fendering system can be so simple and strong. Such structures, however, are now so expensive in initial cost that in future, whenever it is essential to retain earth, steel sheet-piling is most likely to be used. With steel-sheet-piled wharfs fully backed with earth up to coping level, fendering should be just as simple as with concrete

walls and, so far as fenders are concerned, maintenance costs should be equally low.

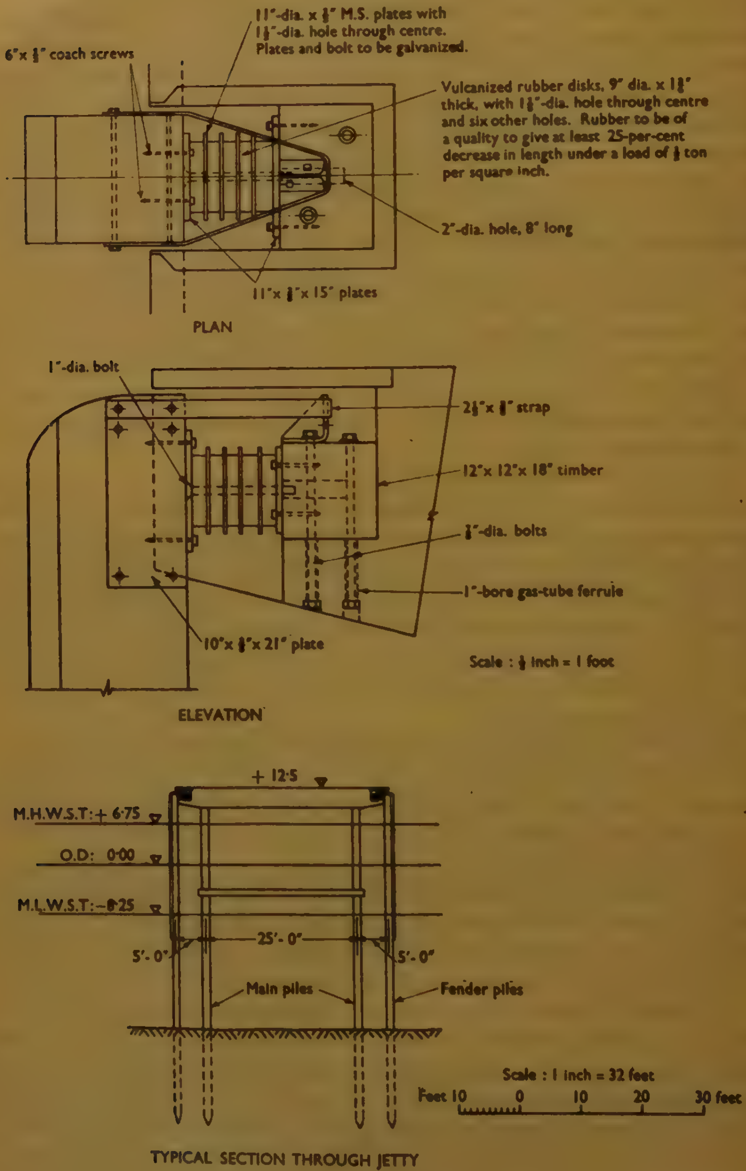
Although steel sheet-piling may be the cheapest way of providing a solid-faced wharf or jetty, by far the cheapest structure, with regard to initial cost, results when the retaining of earth can be avoided and an open piled structure adopted. Fendering, however, then becomes a very real problem ; the open piled structure itself has to be protected against vessels instead of the vessel having to be protected against the berth. Subsequent costs of maintaining fenders are almost certain to be somewhat high, no matter how well they are designed and, if the fenders are not adequate, maintenance costs may prove to be very high indeed. It is doubtful if even the heaviest of maintenance charges could offset the extra initial cost of a gravity structure compared with a piled one, but the fenders of the latter may represent 25 per cent of the total first cost and if the best overall economic efficiency is to be obtained from open structures it will usually be found that the fendering is the most important feature of the design—especially where water depths in excess of 20 feet and vessels greater than 10,000 tons are involved.

Accordingly it is hoped that the following description of twenty different fendering systems may be of interest. All except two have been built during the past 12 years and the descriptions are in chronological order, concluding with designs now under consideration. A large number were used during World War II at a time when substitute materials had to be adopted because more appropriate ones were not available. On the whole, few of those can be regarded as successful by permanent peace-time standards, but results are included since they at least indicate what should be avoided when possible.

No. 1 (*Figs 1*).—This is a pre-war design. The fender pile is of timber and resilience at the head is provided by a rubber spring made up of alternate disks of rubber and steel and specified to compress 3 inches under a load of 50 tons, that is, K.E. = 75 inch-tons. Compression, however, can only take place along a line at right angles to the length of the jetty ; there is no capacity for absorbing glancing blows with a component along the jetty length. In this respect it resembles steel springs and is a typical example of the attempts that were beginning to be made just before the second World War to provide some degree of resilience in fendering.

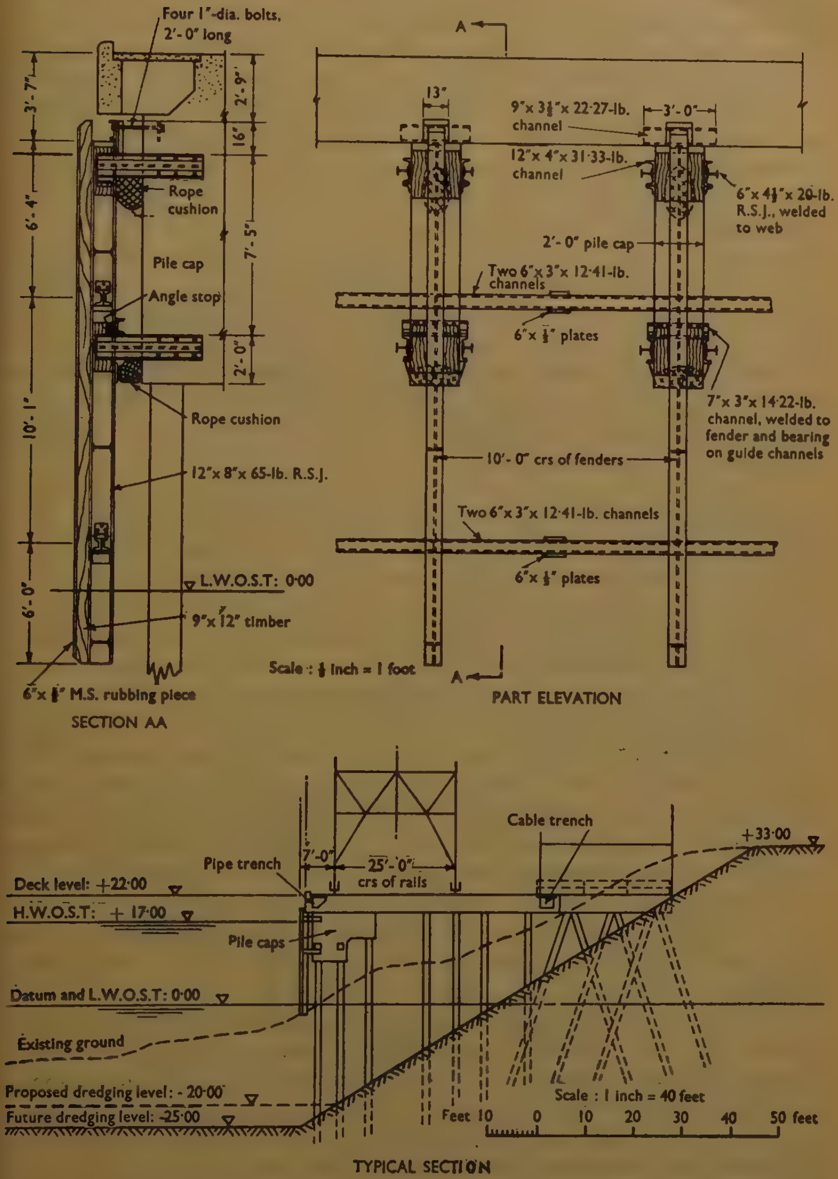
The general usage of the jetty was expected to be restricted to large store barges with occasional heavier vessels of up to about 1,000 tons. Under emergency war conditions it undoubtedly received heavier use than this and adequate maintenance over that period was not possible. During the war the use of rubber for fender units was prohibited, but as soon as the war finished and designing along more permanent lines was allowed, the question of reintroducing rubber was considered. A report on the state and behaviour of the units shown in *Figs 1* was called for and it was learned that not one of them was remaining. Apparently the bolts holding the

Figs 1



Design No. 1

Figs 2



DESIGN No. 2

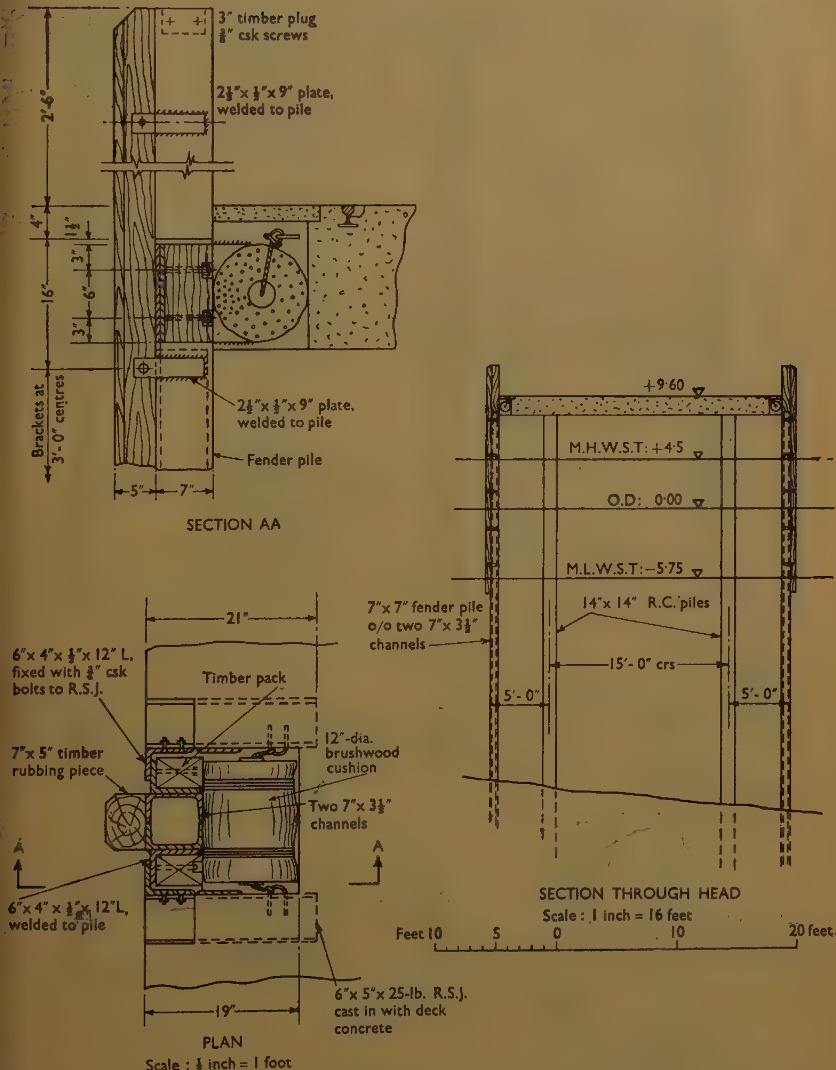
disks together had sheared under the action of glancing blows and the rubber and steel had fallen off and been lost in the water. With better maintenance and less excessive usage, the units might have given better service, but as it was the jetty had only been built about 6 years and, when all allowances were made, the design could hardly be considered successful.

No. 2 (*Figs 2*).—This was a war-time design. Timber and rubber were no longer available and the main fendering was of standard rolled steel sections cantilevering down into low water and free to move at right angles to the jetty against the cushioning effect of rope units. There was no allowance for longitudinal movement and in this direction reliance had to be placed upon the fact that the fendering system was tied together by horizontal runners and should act as a whole unit. In practice it was not a success. The ropes had little or no positive recovery against compression and the steelwork, under the action of continuous blows which induced stresses beyond the elastic limit, gradually assumed permanent sets which added up to excessive distortions. This effect was made worse by the fact that the clear spans of individual members were generally small and the energy-absorption capacity was correspondingly low.

No. 3 (*Figs 3*).—This was a war-time design intended to cater for vessels of less than 1,000 tons, but actually had to take some of 2,000 tons. The fender piles were of a rather light steel section made up from two 7-inch-by-3½-inch channels. Such a pile 40 feet long and loaded at the centre can take a force of 3 tons with a deflexion of 7 inches when stressed to 15 tons per square inch. Hence for blows at low water the kinetic-energy capacity inside the elastic limit is 10½ inch-tons—a not unreasonable value for normal berthing. Resilience against blows at or above high water (certain vessels using this jetty had beltings higher than deck level which explains why the fender piles had to project above the deck) was provided by the brushwood units. After 4 years of heavy use the fendering system was badly damaged. Many of the rubbing pieces were stripped off ; main fender piles were distorted under permanent set in various directions, and most of the brushwood units were either lost or crushed beyond use. If better maintenance had been possible the wear and tear would have been considerably reduced, but basically neither the brushwood nor the fender piles were really strong enough for what they were eventually called upon to do.

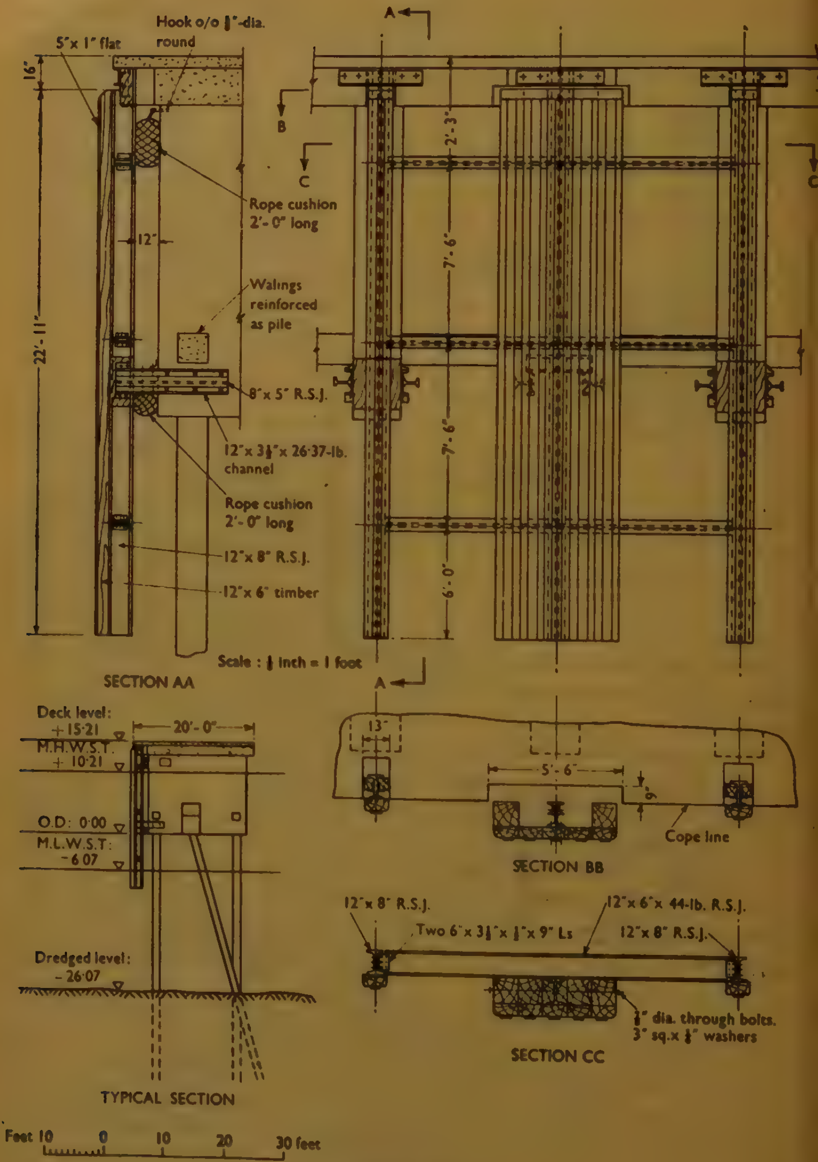
No. 4 (*Figs 4*).—This was also a war-time design on somewhat similar lines to No. 2 except that the fendering system as a whole was not of uniform strength throughout its length, but comprised clumps of extra strength and with some resilience at regular intervals, connected between by somewhat weaker and fixed fendering. The underlying principle of this was that the clumps, being proud of the main line, would receive most of the blows but should be strong enough to deal with them. Paddle-driven tugs with projecting sponsons amidships were among the main users of this jetty and they quickly caused considerable damage to the

Figs 3



DESIGN No. 3

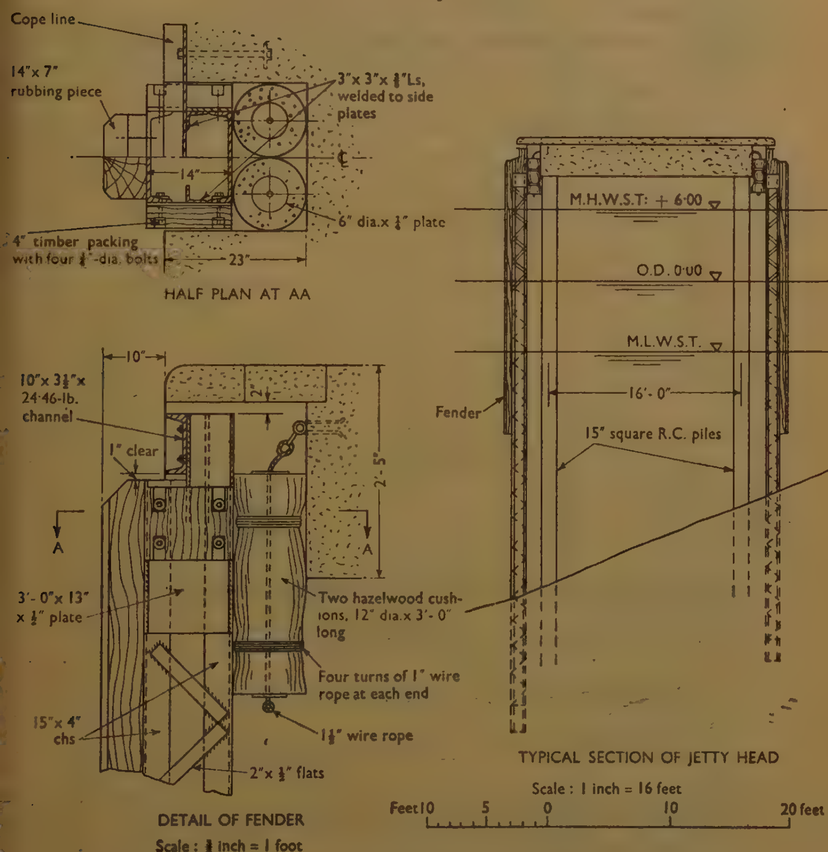
Figs 4



DESIGN No. 4

whole of the fendering—including the stronger clumps. Such vessels with their sponsons are extremely difficult to deal with under any conditions and probably only some form of heavy close-boarded timbering would have been successful. The steel and brushwood design as built failed in the end by permanent set in the brushwood and the steel—mainly about the weaker axis of the latter from longitudinal blows.

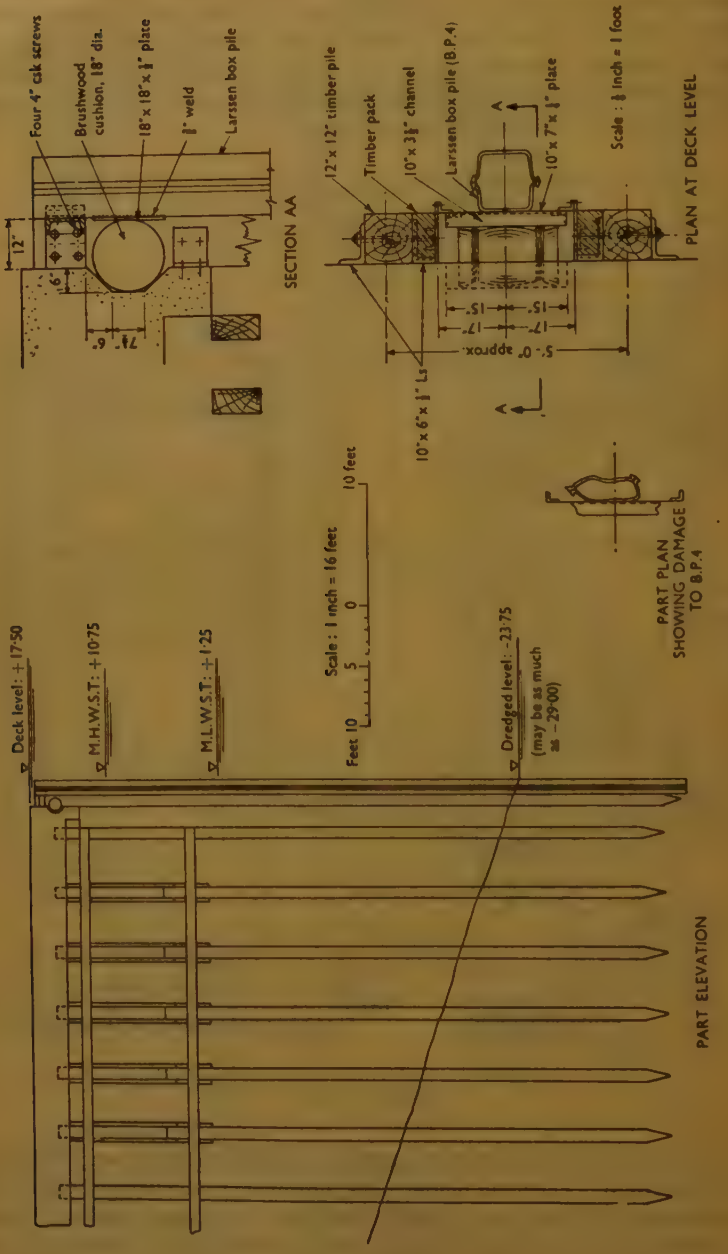
Figs 5



Design No. 5

No. 5 (*Figs* 5).—This was another war-time design and basically similar to No. 3. The steel fender piles, however, were stronger all round; the brushwood fenders were larger; the jetty is in more sheltered water and vessels can be brought alongside with greater ease. Apart from an accident caused by the bow of a submarine nosing its way in behind one

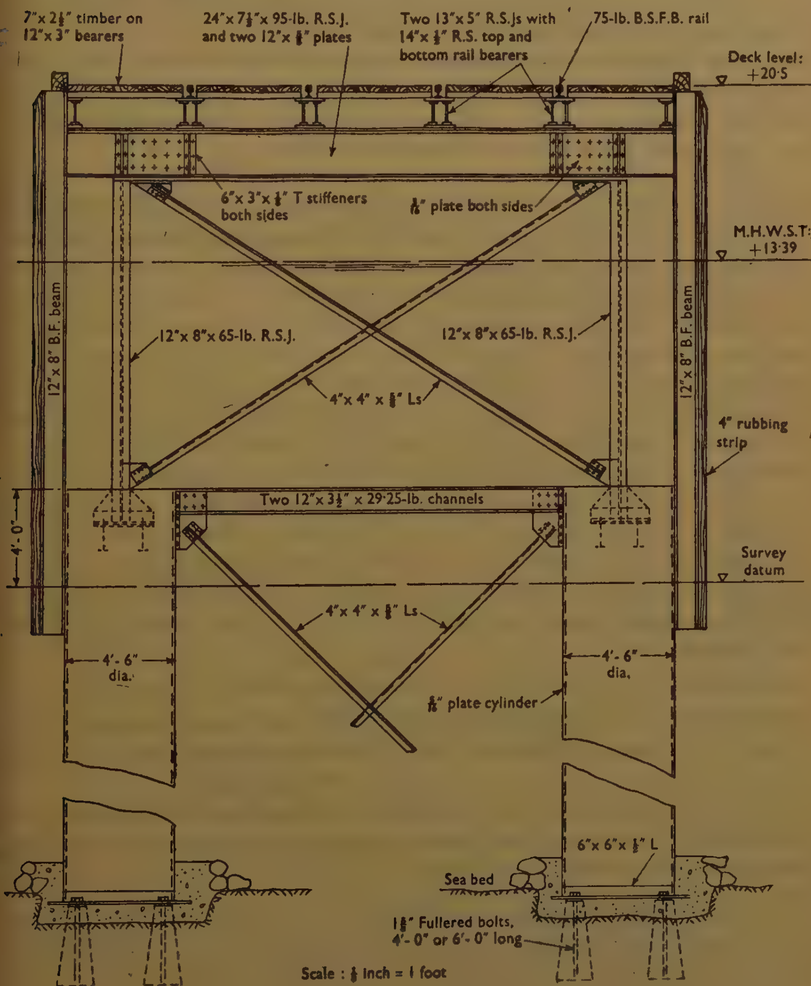
Fig. 6



DESIGN No. 6

of the piles at one end of the jetty, the fendering has given, and is still giving, excellent service with almost negligible maintenance. As such it demonstrates that this type can be quite satisfactory in those rare cases where no parts of it are stressed beyond the elastic limit.

Figs 7



DESIGN No. 7

No. 6 (*Figs 6*).—This, too, was a war-time design, similar to Nos 3 and 5 except that steel box-piles (which have almost equal strength about the XX and the YY axes) were used. Quite heavy vessels—up to 15,000 tons

—used this berth and the main point of interest is that the tops of the piles were left empty when first fixed and the blows from the larger vessels at deck level were sufficiently heavy to crush and distort the hollow piles against the concrete deck. As a result of this experience, these and all subsequent box-piles have been filled with concrete to a distance of 5 feet from the top. Since this has been done the fendering has been satisfactory, but it should be remembered that large wall-sided vessels will usually deliver most of their impact at deck level rather than lower down on the pile. When the crushing tendency is guarded against the fender pile should then be satisfactory, but unless adequate cushioning is provided the jetty structure itself may well be overloaded and, of course, the point-load reaction on the vessel might become important.

No. 7 (*Figs 7*).—This was built in the early days of the war. It was originally intended that the fendering should be of 12-inch-by-8-inch timber, but owing to timber shortage steel joists were substituted and in a comparatively short time these were badly distorted through the continual application of loadings beyond the elastic limit. Quite small vessels were sufficient to cause accumulated deflexions of 12 inches and the problem was discussed as a matter of general interest with Professor J. F. Baker at Cambridge. He pointed out that the joist with a span of 13 feet and having both ends fixed would carry a central load of 12·55 tons at a stress of 15 tons per square inch and a deflexion of 0·29 inch, giving an elastic kinetic energy value of 1·82 inch-tons—say 2·0 inch-tons. But since the joist would be bending about its YY axis it would have a shape factor approaching 1·5, so that the "failing" load was at least $12{\cdot}55 \times 1{\cdot}5 = 18{\cdot}8$ tons, giving an elastic K.E. value of about 4·0 inch-tons. For a deflexion of 12 inches, however, and assuming the idealized form of load/deflexion diagram shown in *Fig. 8* (*a*), the K.E. value would be 221·5 inch-tons, that is:

$$\tfrac{1}{2} \times 18{\cdot}8 \times 0{\cdot}43 + 18{\cdot}8\,(12 - 0{\cdot}43) = 4{\cdot}0 + 217{\cdot}5 = 221{\cdot}5 \text{ inch tons}$$

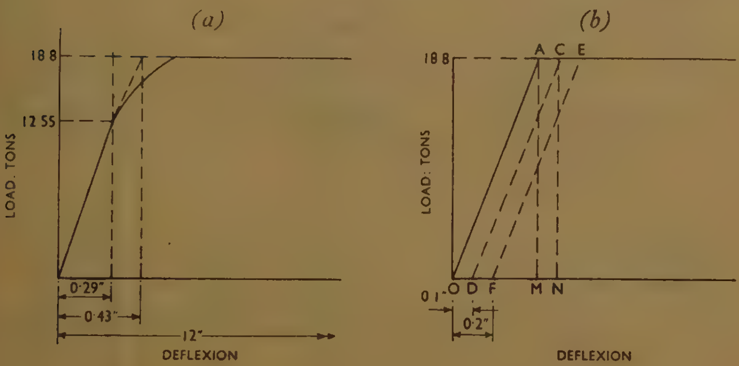
The difference between the elastic and the ultimate kinetic-energy capacity of steel is, of course, well known, but it is felt that this numerical example of 4 inch-tons and more than 220 inch-tons is interesting.

It was known from the site, however, that no single blow had caused the deflexion of 12 inches and Professor Baker's actual comments on the effect of a succession of blows were:

> "with regard to your question about successive blows, the position is as follows assuming in *Fig. 8* (*b*) as before (*Fig. 8* (*a*)) an idealised load deflection curve which is not far from the truth. If each successive blow transmits an energy of less than OAM, i.e. 4 inch-tons then they can be repeated indefinitely without a permanent set. If, however, any blow transmits a greater energy than this, a permanent deflection will result. Suppose, for instance, that a ship bumps the column transmitting 5·88 inch-tons then the energy absorbed

by the column will be represented by OACN (*Fig. 8* (*b*)) AC being 0·1 in. When the ship releases the column a permanent set OD magnitude 0·1 in. will remain. After this the column could once more take an indefinite number of bumps transmitting less than 4 inch-tons (or perhaps a little more if we took into account every refinement, as strain age hardening would put up the yield stress of the material by an amount which can be safely ignored). Under such a bump transmitting 4 inch-tons the column would deflect, as represented by DC (*Fig. 8* (*b*)) the K.E. being represented by triangle DCN. On

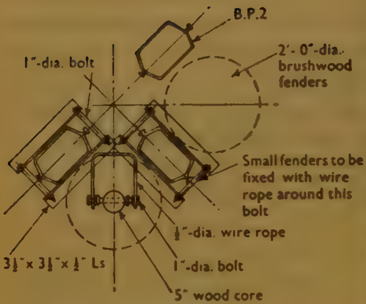
Figs 8



being released, the column would return along CD to its permanent set D. However, any bump greater than 4 inch-tons would cause further permanent set so that if there was another transmitting 5·88 inch-tons the column would deflect along DCE and on release would be left with a permanent set of 0·2 in. and so on. You will see, therefore, that to produce the same final permanent set the sum total of the work done by a number of small blows can be many times greater than that done by one big blow."

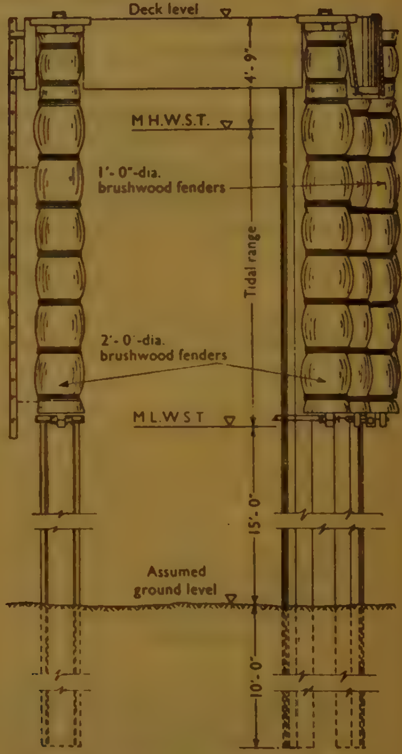
This particular fender is not very significant in itself, but it is described in detail because it is a good example of the basic principle underlying the action and possible failure of all steel fendering. Although, as Professor Baker points out, the sum total of work done by a number of small blows may be very large, yet an individual blow greater than 4 inch-tons and therefore capable of producing a permanent set is not high. A barge of 100 tons bumping at $\frac{1}{2}$ foot per second would give it, and in a short spell of choppy weather it might well be only a matter of a few days before a single barge lying alongside had applied sufficient bumps for the accumulated sets to be appreciable. In fact, this is what actually happened at the site under consideration. If the fenders had been of 12-inch-by-8-inch timber as originally intended, then for a working stress of 1,400 lb. per square inch

Figs 9



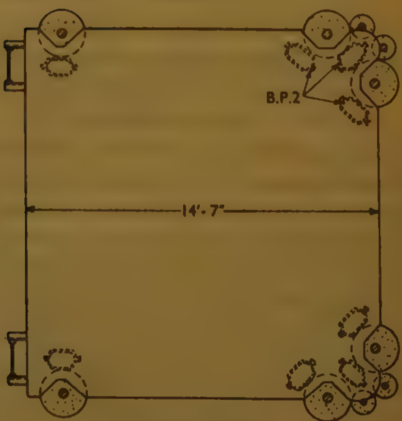
PLAN SHOWING FIXING OF FENDERS AT BOTTOM

Scale : ¾ inch = 1 foot



Scale : ⅛ inch = 1 foot

ELEVATION



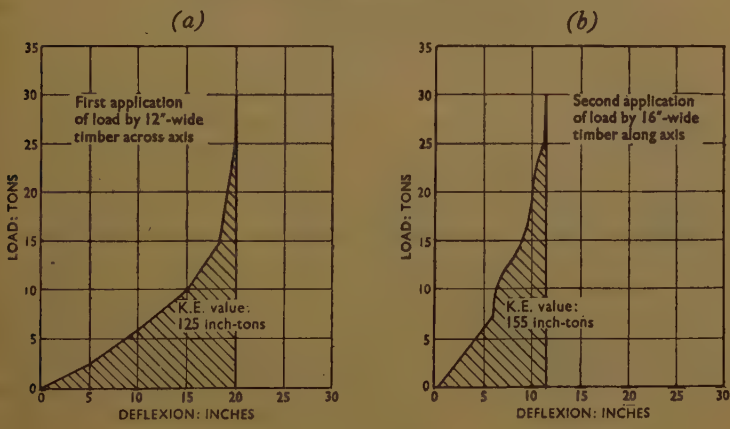
PLAN ON DECK

DESIGN NO. 8—EIGHT-PILE DOLPHIN

the central load would have been 4·0 tons with a deflexion of 0·3 inch and a K.E. of 0·6 inch-ton. For a failing stress of, say, 5,600 lb. per square inch the K.E. would have been 10·0 inch-tons, since K.E. is proportional to the square of the stress. Although a stress of 5,600 lb. per square inch would never be adopted as a working stress for timber it is not too high an assessment for a failing stress and in the elastic limit the timber would have been at least twice as resilient as the steel, and might well have been just strong enough to take the barges. (These timber calculations are based on a span of 13 feet, with both ends fixed ; a central longitudinal blow being about the weaker axis ; and Young's Modulus $1{\cdot}18 \times 10^6$ lb. per square inch.)

No. 8 (*Figs 9*).—This was the largest of various similar dolphins built towards the end of the war for the invasion. Large brushwood fenders of the type shown were used at Southampton before the war, slung in a

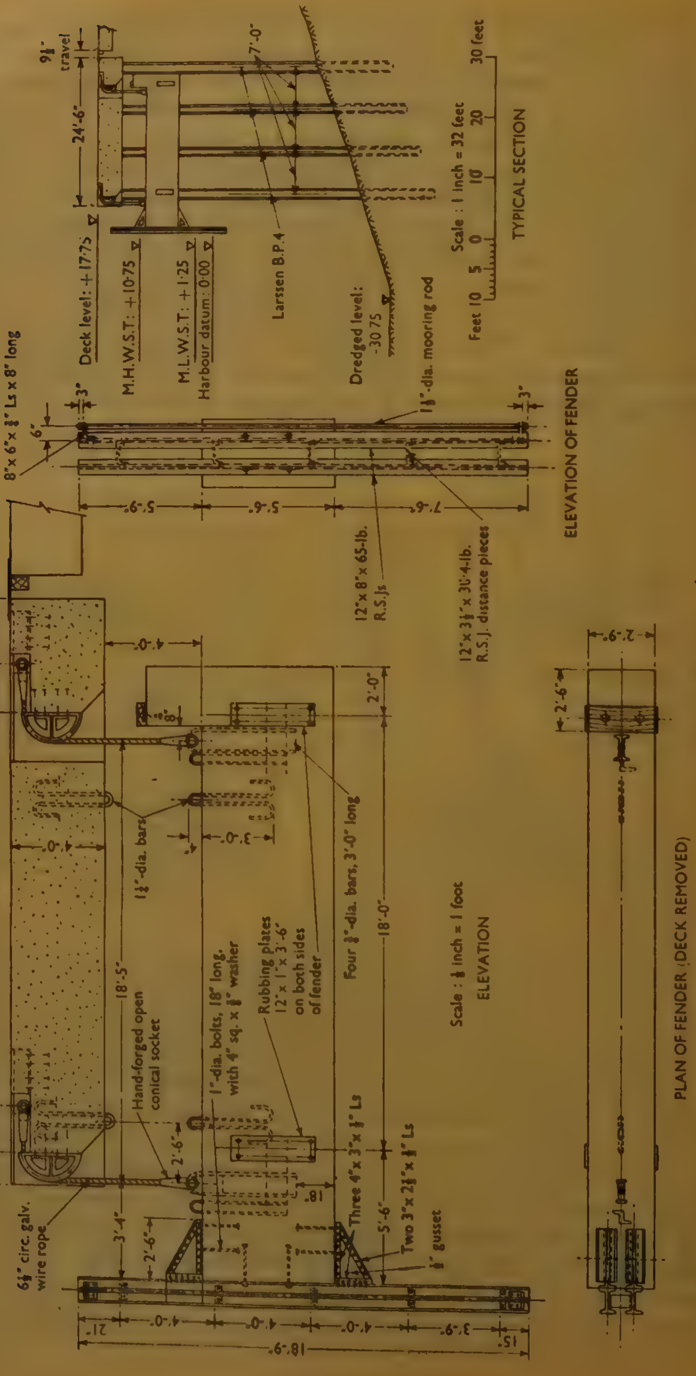
Figs 10



horizontal line in front of the solid monolith wharf in order to protect the relatively weak sides of transatlantic liners against the concrete.

In connexion with the various dolphins, deflexion tests were carried out in a workshop on a fender 2 feet 6 inches in diameter and 9 feet long, with the result shown in *Figs 10* (*a*) and (*b*). For a single blow, the kinetic-energy capacity, as expected, is high and in the workshop it was possible to recover the shape by rolling the fender round the floor. In actual use, however, recovery would not be good. If they were slung horizontally on a chain in front of a solid wall, rotation by wave action might be fair, but since in these dolphins the fenders were slung vertically they were a little disappointing. The principle of having the fenders cradled, as it were, between pairs of piles was sound, and when the fenders were hit for the first time the energy-absorption capacity was quite considerable.

Figs 11



Design No. 9

Unfortunately, however, vessels often came alongside awkwardly and hit the concrete slab direct between fenders and without touching them at all.

This berthing requirement was, of course, a very special one, being entirely operational and temporary. The dolphins generally fulfilled their purpose and the brushwood fenders had a potential capacity which in the few cases where they were struck proved invaluable, but the peace-time application for permanent works seems very limited—especially this use of the brushwood fenders, unless it can be certain that berthing will always take place on them.

No. 9 (*Figs 11*).—This fender consists of a large clump of reinforced concrete slung in a horizontal position from the underside of the jetty deck and free to swing back and lift upwards under the influence of berthing forces applied to the timber rubbing piece. It is essentially a gravity device and quite large K.E. capacities can be achieved with it. The one illustrated weighs 25 tons in air and at low water absorbs 425 inch-tons when pushed right back. Eighteen of them were incorporated in a berth built in 1944. They were distributed in three groups of six along the berth, each group being spanned by a heavy floating fender in front of it so that the six clumps could be relied upon to act as one unit with a combined K.E. value of 2,550 inch-tons. In addition, at the maximum movement the dolphin structure itself was calculated to be under a thrust of 130 tons with a deflexion of 9½ inches, giving a further K.E. of 600 inch-tons.

Although the berth was built during the war, it was expected to have a peace-time value and the design was regarded as suitable for a permanent structure ; in any case, vessels of up to 20,000 tons had to be catered for. Advantages claimed for it are that it is a simple gravity mass of concrete that can be built between tides and that it is suspended entirely above high water, thereby avoiding all tidal or underwater bracing to the main structure. A possible disadvantage is that it had no longitudinal energy capacity—blows can be absorbed only at right angles to the line of the berth. With very large vessels, berthing must always be undertaken with extra care and apparently they usually do come alongside with almost negligible forward speed. In addition, it was known that large floating fenders would always have to be used with this berth and these themselves can cater for a certain amount of longitudinal movement. It will be noted that the side clearance between concrete clumps and main piles is 4¼ inches.

So far, during the 7 years they have been in operation, they have behaved quite satisfactorily. No excessive wear on slings or moving parts has been observed ; maintenance costs have been nil ; lateral movements under wave action have not been excessive and on the few occasions that 15,000-ton vessels have used them, they have moved back and absorbed blows as required. The fittings for each clump (that is, two wire strops, two anchors, and two grooves for wire strops) cost about £200 in 1944

and the concrete about £500, making a total of £700 for each clump with a K.E. value of 425 inch-tons.

No. 10 (*Figs 12*).—This is a post-war design, very similar to No. 9, which has been incorporated in two different berths, each of which has to provide for future vessels of up to 40,000 tons. The only real difference between Nos 9 and 10 lies in the method of suspension. An opinion had been given that the wire strop in design No. 9 might need fairly frequent renewal, and since the initial cost of £100 each (including grooves and anchors) seemed rather high they were replaced in design No. 10 by steel links coupled with turnbuckles for site adjustment.

The first berth in which these were used was somewhat exposed. Waves 4 feet high are frequently experienced for spells lasting several days and these set up so much lateral swinging that the fenders gave serious trouble before the berth was even completed. When assembling the clumps, the coupling nuts were so stiff that a stout tommy bar and a 7-lb. sledge hammer had to be used on them. Two men could not move the tommy bar by hand. But in rough weather, wave action was sufficient to undo the nuts in a few hours. Also the continual thumping of the clumps against the main jetty piles was so severe as to be quite unacceptable and the links showed a distinct tendency to jump off the trunnions. Although the original side clearance between clump and main piles was only 2½ inches, compared with 4½ inches for No. 9, this was enough to allow a build-up of considerable side-sway momentum, the amount being noticeably greater under certain conditions of wave frequency. The trouble was completely cured in the end by closing the clearance down to less than ½ inch. This was adopted only as a last resort because it had been felt that the larger clearance would provide a certain amount of lateral resilience. Since in this at-rest position, however, the clumps are at "bottom dead centre," the amount of resilience provided by a 2½-inch movement is probably quite insignificant.

In view of this trouble at the exposed berth, the second fenders of No. 10 design were constructed from the beginning with the ½-inch clearance, but since this site was relatively sheltered it might not have mattered if the larger clearance had been adopted.

No. 11 (*Figs 13*).—It was fully realized that the rope-and-brushwood springs that had to be used during the war were only makeshift devices, and when the war ended and rubber became available again, consideration was given to its use. Reports were called for on the behaviour of pre-war design No. 1, and since these had failed under longitudinal blows it was obvious that some completely new approach was wanted. About that time a book, "Rubber in Engineering," was published [1] and from a study of this it seemed that the use of rubber in shear offered possibilities. Discussions with certain commercial firms followed—particularly Dr Naunton

[1] "Rubber in Engineering," H.M.S.O., 1946.

Figs 12

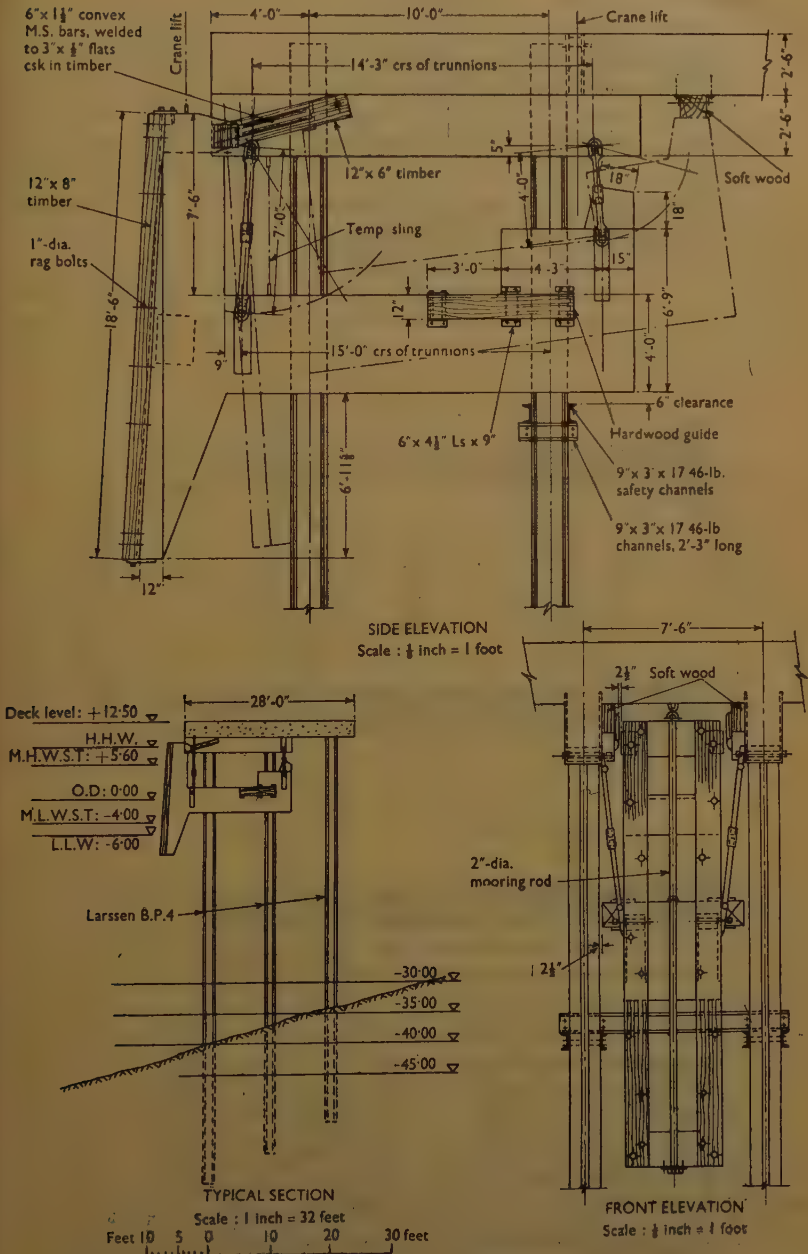
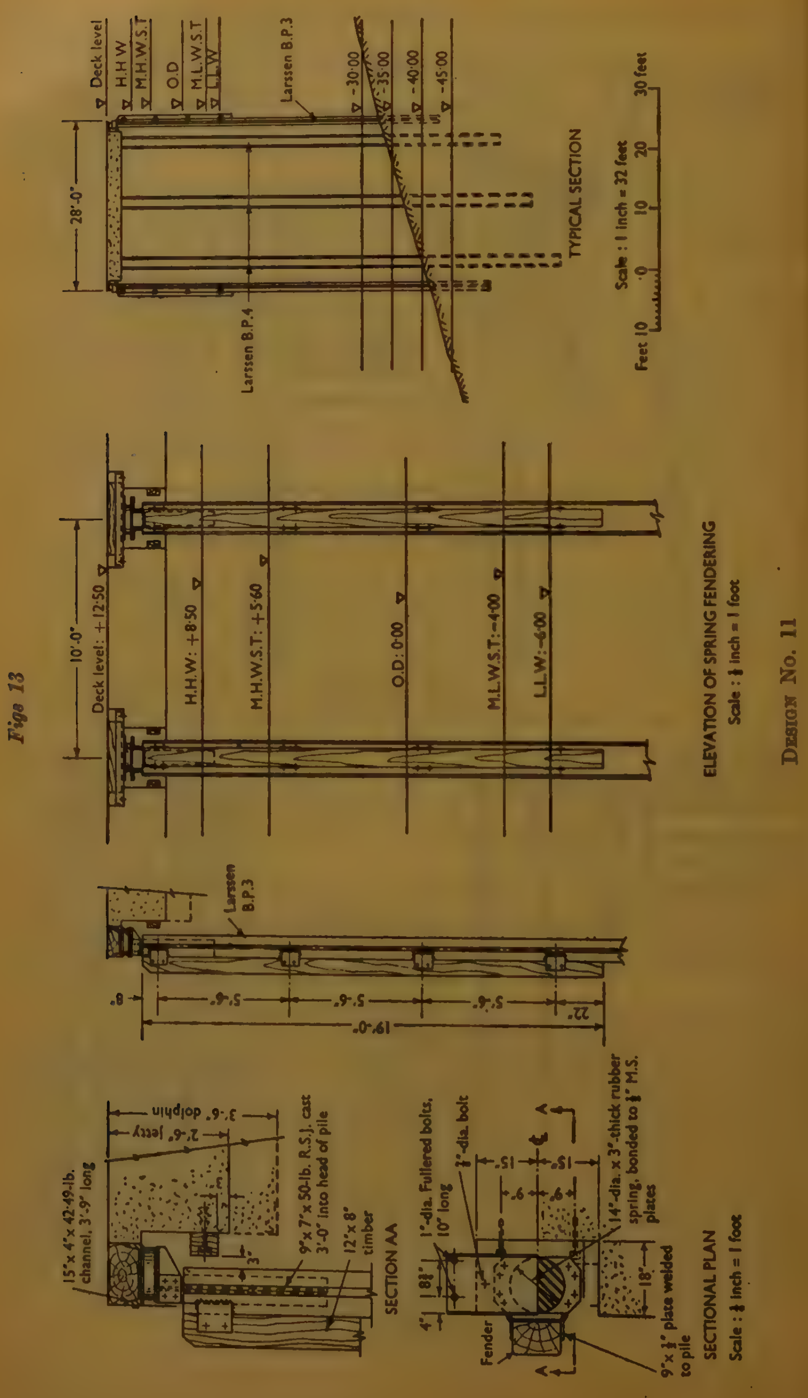

6"x 1½" convex M.S. bars, welded to 3"x ½" flats csk in timber
Crane lift
4'-0"
10'-0"
Crane lift
14'-3" crs of trunnions
2'-6"
2'-6"
12"x 8" timber
12"x 6" timber
Soft wood
1"-dia. rag bolts
7'-6"
7'-0"
Temp sling
5"
4'-0"
18"
18"
15"
3'-0"
4'-3"
6'-9"
18'-6"
12"
4'-0"
9"
15'-0" crs of trunnions
6" clearance
6'-11⅝"
6"x 4½" Ls x 9"
Hardwood guide
9"x 3"x 17 46-lb. safety channels
9"x 3"x 17 46-lb channels, 2'-3" long
12"
SIDE ELEVATION
Scale : ⅛ inch = 1 foot
7'-6"
2½" Soft wood
Deck level: +12·50
28'-0"
H.H.W.
M.H.W.S.T: +5·60
O.D: 0·00
M.L.W.S.T: -4·00
L.L.W: -6·00
2"-dia. mooring rod
Larssen B.P.4
1 2½"
-30·00
-35·00
-40·00
-45·00
TYPICAL SECTION
Scale : 1 inch = 32 feet
Feet 10 5 0 10 20 30 feet
FRONT ELEVATION
Scale : ⅛ inch = 1 foot

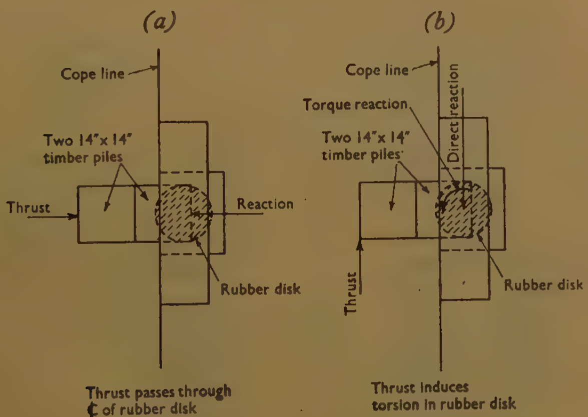
Fig. 13



DESIGN No. 11

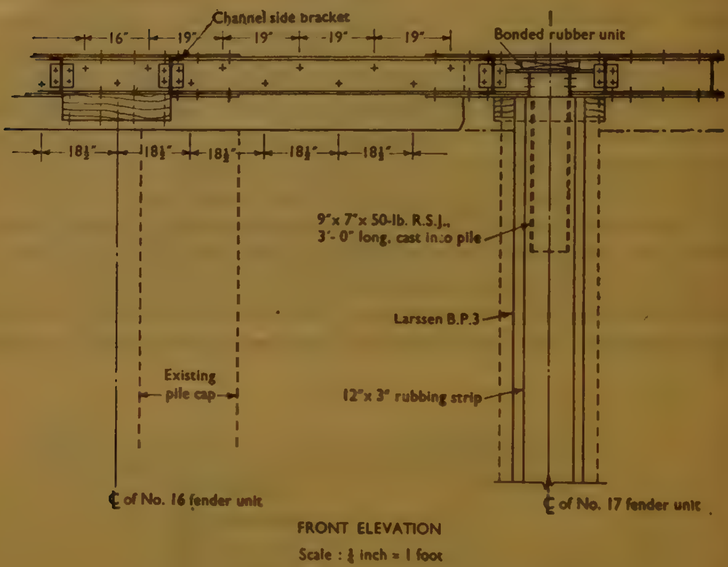
of I.C.I. and Mr Marsh of the Andre Rubber Company—and gradually design No. 11 was evolved. This consists of a round disk of rubber, 14 inches in diameter and 3 inches thick, bonded top and bottom to steel plates, the top plate being secured to the jetty deck, and the bottom plate, through a joist dowel, to the fender pile. The details of the top and bottom fixings are of no special significance and can be adapted to any form of deck or pile (for example, designs Nos 12, 13, and 16). The essential feature is the placing of the rubber in a horizontal plane so that it can provide shear resilience against a horizontal blow or pull applied at any angle. This was made possible by the vast development of bonding technique during the war, it now being an accepted fact that rubber can be bonded to metal strongly enough to guarantee that the bonded joint will be as strong or stronger than the rubber itself. For mechanical vibrations of high frequency, only small angular shear movements should be allowed,

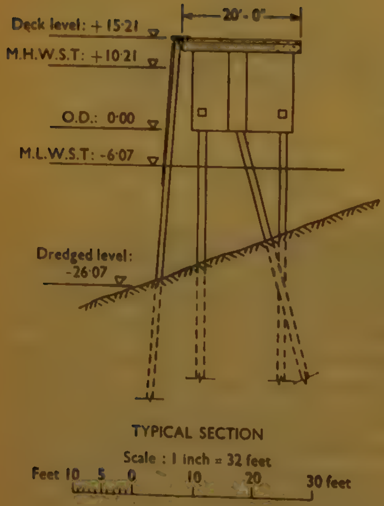
Figs 14

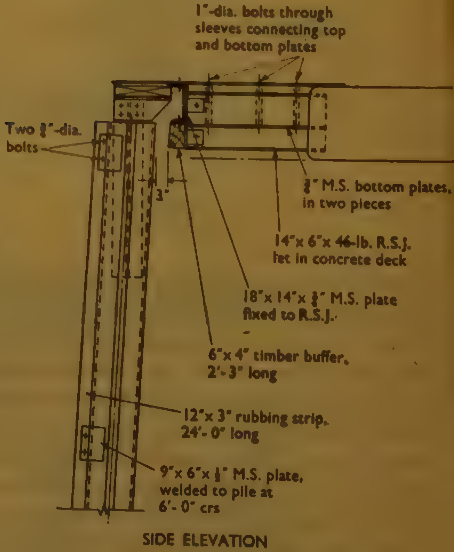


otherwise the character of the rubber will quickly alter. But for the occasional excessive blows that fenders have to sustain, it is acceptable to work to an ultimate K.E. capacity based on shear movements of 45 degrees. This means, of course, that the maximum fender-pile movement to allow for is the same as the thickness of the rubber disk. Brief calculations and some typical K.E. values for different units are given in the Appendix. From this it will be noted that single units cannot give really high K.E. values, but they are extremely neat and simple; they do move in all directions and they do recover. Their one very definite weakness, however, is that they cannot withstand much torsional or twisting shear. If the direction of blows can be made to pass through the centre of the disk (*Fig. 14* (*a*)) there should be no trouble, but where thick rubbing pieces or twin timber fender-piles are used, the old bugbear of glancing blows may well induce sufficient twisting to cause failure in the rubber (*Fig. 14* (*b*)).

Figs 15

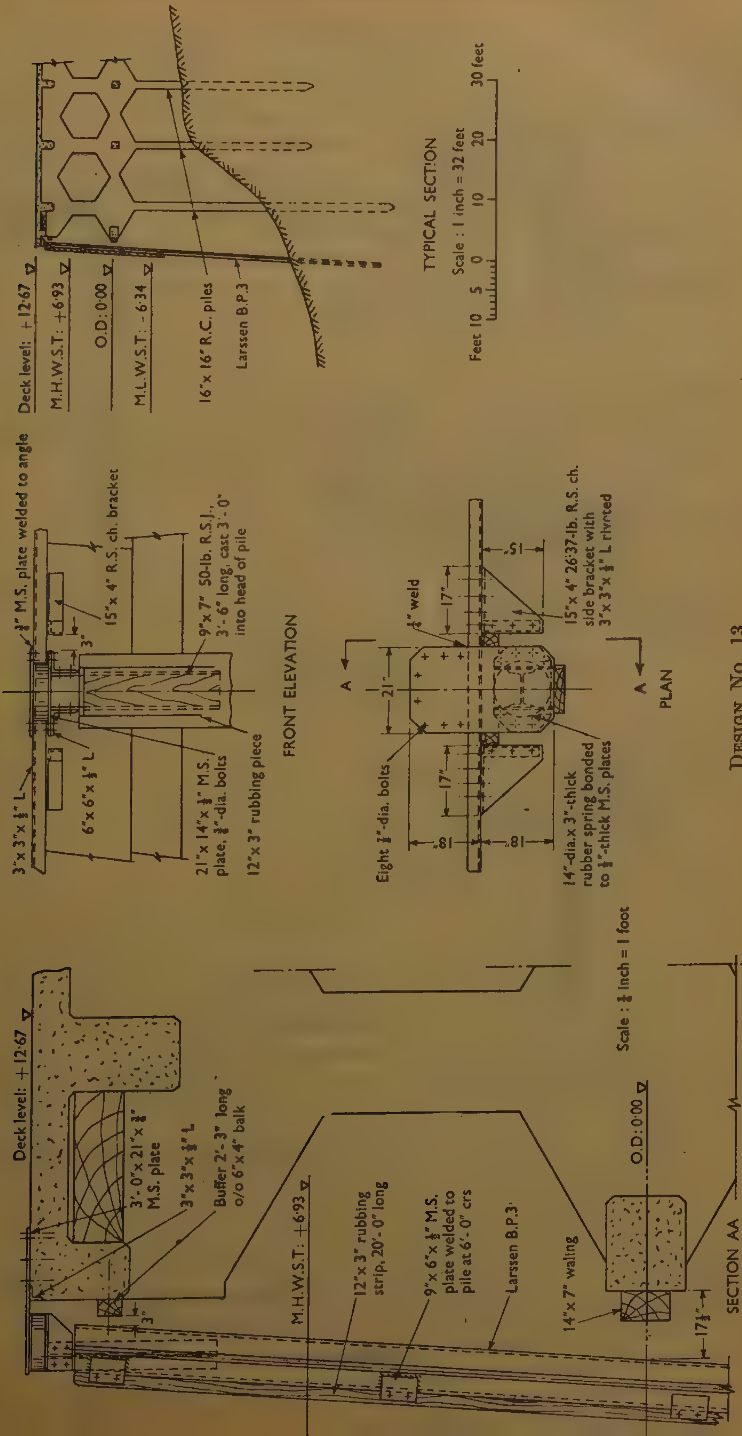






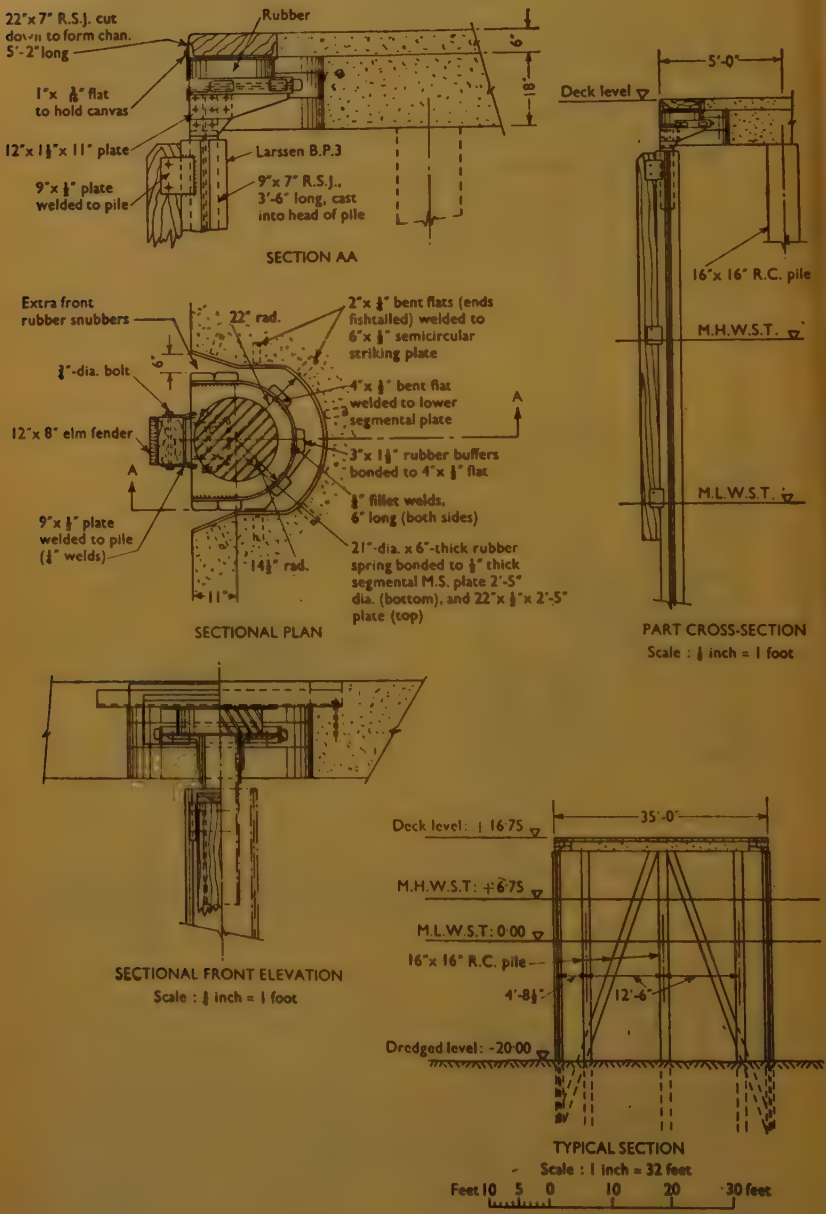
Design No. 12

Figs 16



DESIGN No. 13

Figs 17



Design No. 14

The units have to be made in special steel moulds in which both plates and rubber are subjected to high temperature and pressure. Moulds are expensive, but hundreds of units can be made from one mould and if enough units are required the mould cost can be ignored. It pays, however, when only a few units are wanted, to use a standard type whenever possible, in order to avoid the high first cost of a new mould.

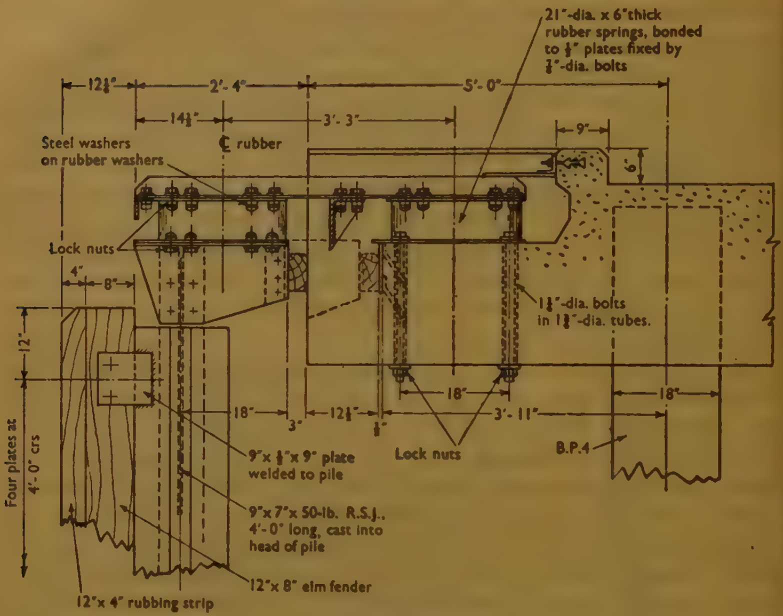
No. 12 (*Figs 15*).—This 14-inch bonded-rubber unit is the same as No. 11, but has been adapted for use on an existing jetty and is, in fact, a repair replacement to No. 4. To date, the fendering has been in use by small craft—up to 1,000 tons—for nearly a year and is quite satisfactory. A minor nuisance is that mooring ropes from jetty bollards down to vessels tend to get caught under the projecting brackets to which the top plates of the rubber units are secured.

No. 13 (*Figs 16*).—This is a single fender-pile and 14-inch bonded-rubber unit fitted to an existing jetty purely as a full-scale test of the rubber unit. The pile is raked outwards to line up with the other fenders on the jetty, but in spite of this rake the bows of a vessel with a considerable flare struck the upper fastening plate, holding the rubber unit, without even touching the pile. As a result, the fastening plate and rubber unit were stripped right off without the latter coming into play at all as a resilient fender. Partly under torsion and partly under shear the body of the rubber was split in two, failure occurring in the rubber and not in the bonding. Seven out of the eight bolts securing the top plate to the deck were sheared through, so it was hardly surprising that the rubber failed too. Considered as a test on the rubber unit proper, this result was really useless—except that it did confirm that the bonding was the strongest part of the rubber—but it did serve to demonstrate that for vessels with flared sides it is essential that the rubbing face of the fender pile should be carried right up to the level of the coping. Reference to *Figs 13*, *15*, *16*, and *17* will show that with these designs this cannot be achieved. Main piles must stop a little distance below the bottom plate of the rubber unit; the timber rubbing pieces can be carried right up to the level of the bottom plate, but this must still be some inches below deck level.

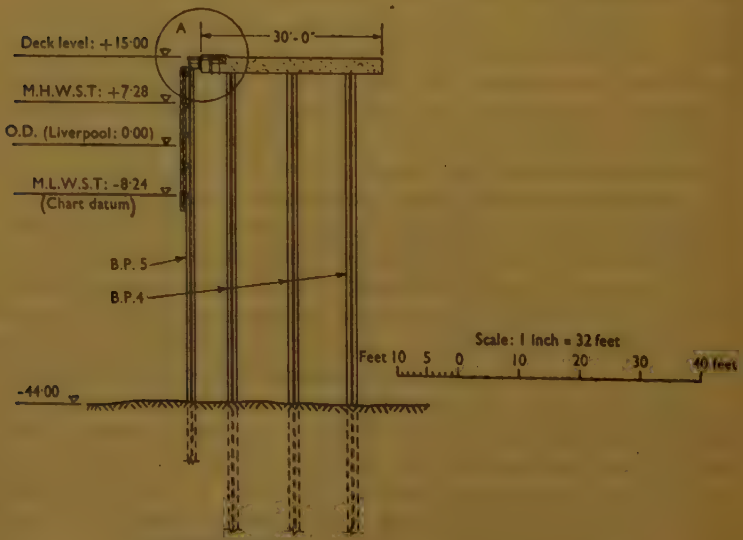
No. 14 (*Figs 17*).—This is a larger bonded-rubber unit built into a new jetty for vessels of up to 2,000 tons. The estimated ultimate K.E. value of the unit is 75 inch-tons, so that each fender pile should be capable of dealing with half the weight of a vessel at a speed of just more than $\frac{1}{2}$ foot per second. Since the vessel might reasonably be expected to strike more than one pile-fender—piles are at 10-foot centres—the K.E. provision is possibly a little generous, but in view of the operational nature of the jetty this was done deliberately. By the time this design came to be made, experience of the torsional weakness of rubber was available and extra front snubbers of solid rubber were added and the shape of the housing in the deck amended in order to limit the torsional deflexion.

No. 15 (*Figs 18*).—There are eight bonded-rubber units similar to

Figs 18 (a)



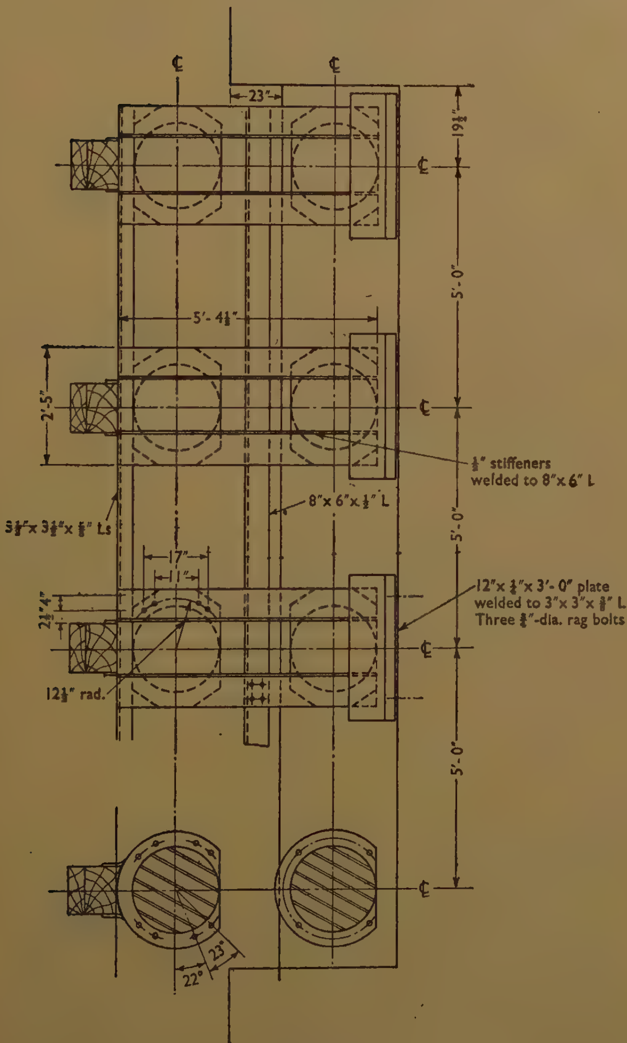
ENLARGED DETAIL AT A
Scale: ⅛ inch = 1 foot



TYPICAL SECTION

DESIGN No. 15

Fig. 18 (b)



PLAN OF FENDER GROUP
Scale: ¼ inch = 1 foot

DESIGN No. 15

design No. 14 in this fender. The rubber units are arranged with pairs in series in order to double the deflexion and the four pairs are framed together so that the whole group will act as one. With a movement of 12 inches and a load of 100 tons (4 × 25 tons) the total K.E. value is 600 inch-tons, and with four groups distributed along a berthing head 200 feet long the total K.E. capacity is 2,400 inch-tons—sufficient to take the full weight of 40,000-ton vessels moving at ½ foot per second.

A group built to this design, therefore, has a K.E. value comparable to that of a single concrete clump in design Nos 9 or 10, and if used in conjunction with a dolphin similar to design No. 8, but with about twice as many piles, the dolphin itself would probably deflect 6 inches under the 100-ton blow, giving a total movement—dolphin and rubber fender—of 18 inches and a K.E. value of 900 inch-tons.

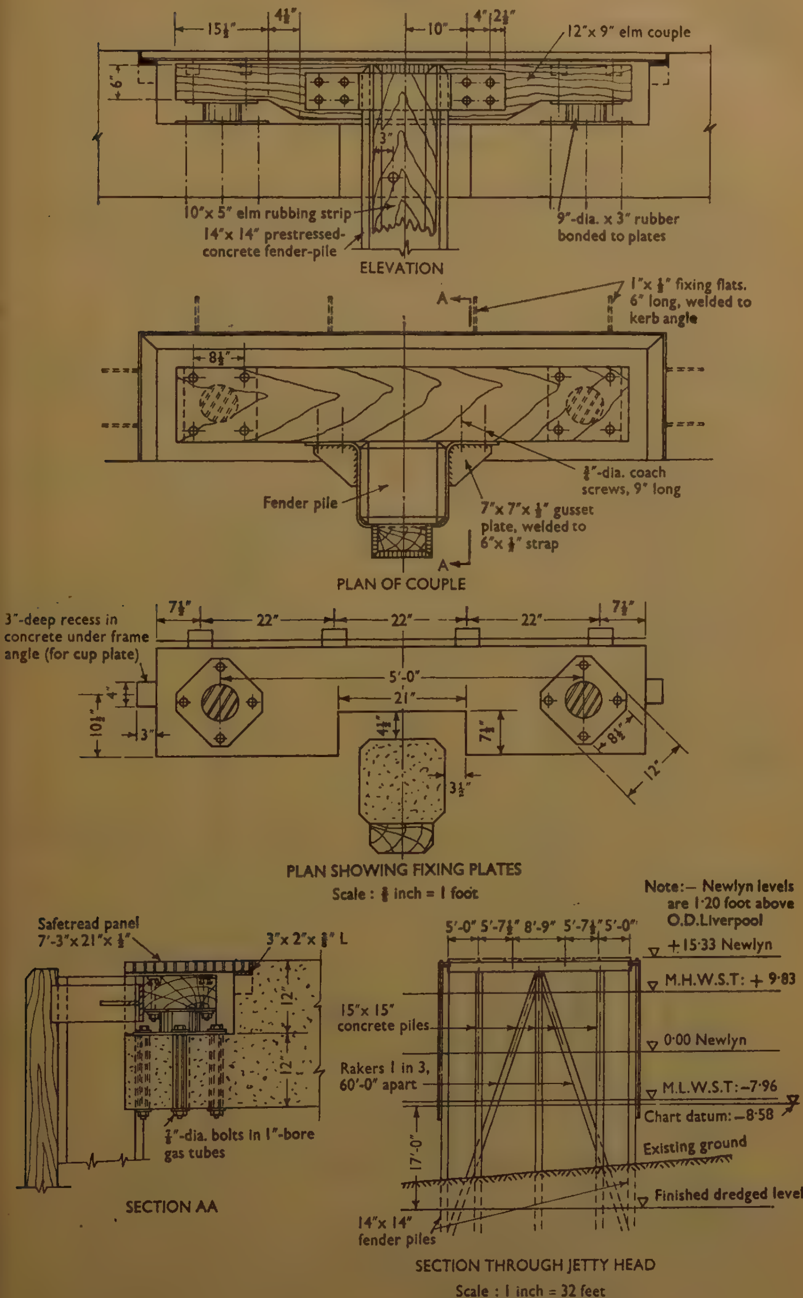
Where the fender piles are long, however—they were 100 feet in length in the cases under consideration—the cost of building a bonded-rubber group like this is more than that of a concrete clump, but the rubber group does have longitudinal resilience and it can be built after the main deck concreting is finished without having caused delay to that concreting.

No. 16 (*Figs 19*).—This is another later development of bonded-rubber units, again used in pairs entirely for the purpose of eliminating the torsional effect on single units. The K.E. value is quite low, being about 12 inch-tons, and the design is suitable only for light duty. Timber is used for the cross-piece between the units so that site fitting will be easier and so that the prestressed concrete fender-piles can be readily bolted to it. These prestressed concrete piles were specified to have an ultimate bending resistance of 60 tons-feet, and it will be noted that they are carried up to coping level.

Nos 17, 18, and 19 (*Figs 20, 21,* and *22*).—Fender No. 17 was erected soon after the first world war as a repair job to a fairly typical heavily braced steel jetty with a very high tidal range. The main fender-pile is a composite steel section made up of standard steel channels and plates. It has good strength about the XX axis, and some extra resilience about that axis was provided at the head by a rope cushion. The strength about the YY axis—which has to resist longitudinal blows—is not so good (but not really low) and the rope cushion offers no resilience at all. Vessels of up to 4,000 tons used the jetty and after World War II the fendering was in a rather bad state, timber rubbing pieces having been torn off and the main pile-members being badly distorted. An inspection confirmed what might be expected, namely, that distortion was generally about the YY axis and worse in the upper lengths of the piles. This suggested that longitudinal walings were required to distribute longitudinal blows over a number of piles, and design No. 18 was evolved, in which groups of piles are framed together to act as a whole unit and bonded-rubber units are inserted at the head to give added resilience there.

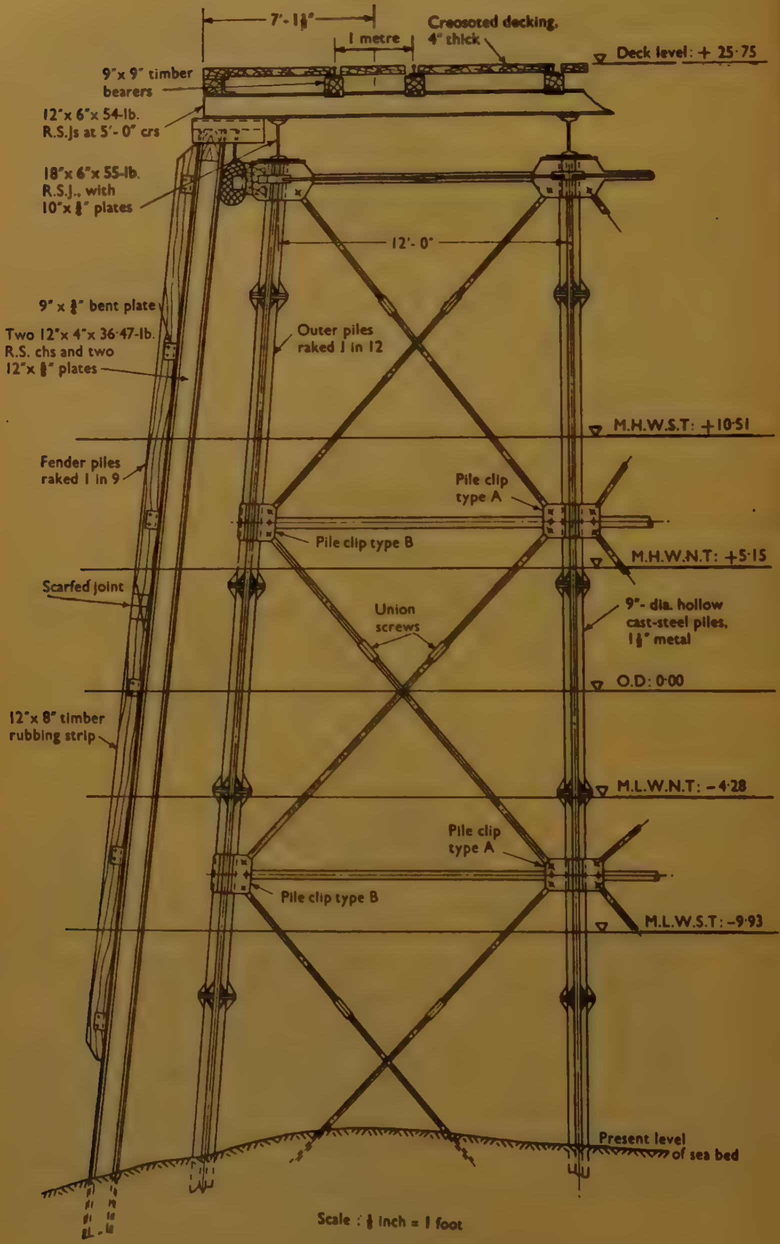
The users of the jetty had claimed that a berthing speed of 1 knot

Figs 19



Design No. 16

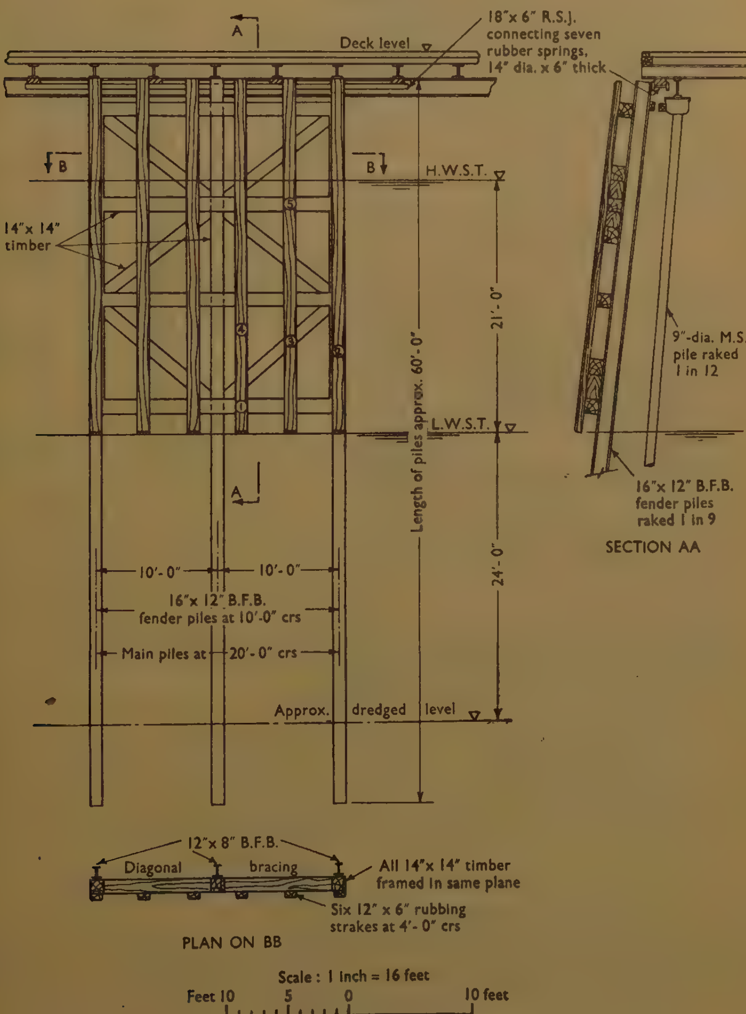
Fig. 20



Design No. 17

should be catered for from vessels of 4,000 tons. If half the weight of the vessel were assumed to act in one blow and if a speed of 1 foot per second were adopted in lieu of 1 knot, the K.E. to be absorbed would be about

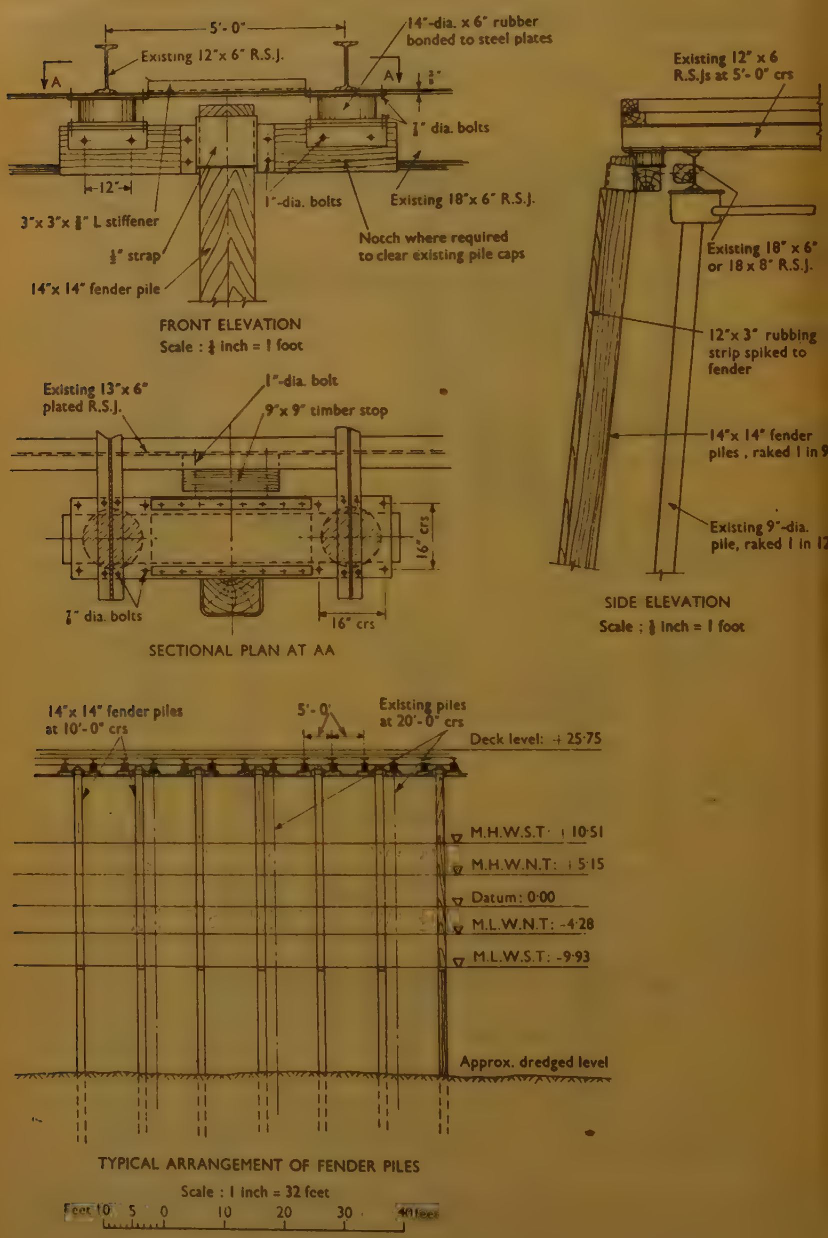
Figs 21



DESIGN No. 18

400 inch-tons. This is quite high and to take it in a longitudinal direction it is essential, with design No. 18, to frame all the main piles into a truss with diagonal bracings. It was thought that timber diagonals would ease

Figs 22



DESIGN No. 19

construction problems, since timber could be readily trimmed to size on site. Even so, the design was cumbersome and costly and not really satisfactory. The group of four piles could not be expected to act as a whole unless struck a square central blow. When the fender is analysed for isolated point loads at the positions numbered in circles (*Figs 21*), it soon becomes obvious that there are internal weaknesses within the framework itself and that for most of these points of loading only a small portion of the potential K.E. can be developed. Also, the seven 14-inch-diameter bonded-rubber units required a total force of 77 tons to deflect them 6 inches and it is doubtful if the deck system of the jetty could safely take this load.

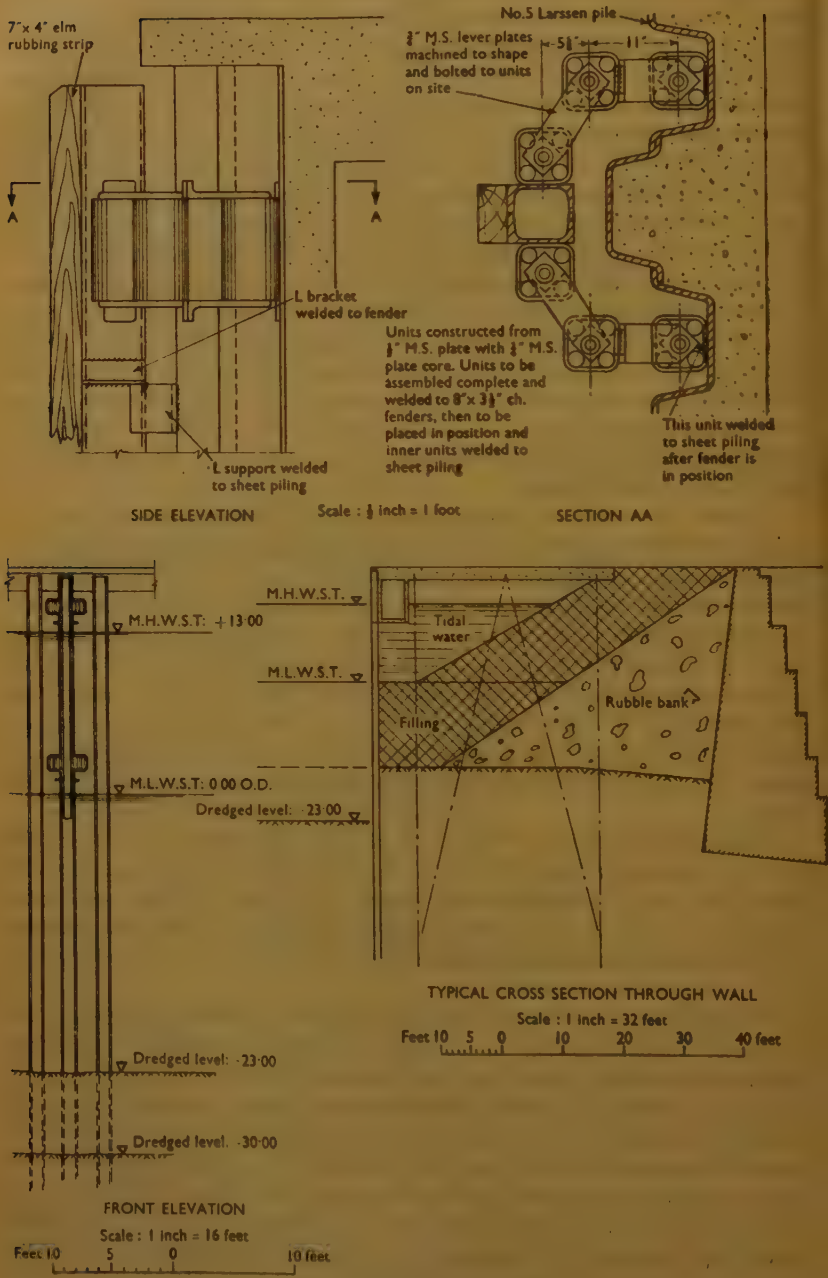
Concurrently with these calculations, Table 1 (pp. 78, 79) was drawn up giving comparative properties of various possible fender piles, taken as being 60 feet long with simple supports and point-loaded at the centre. From this it is seen that the pile that had failed was quite resilient about the XX axis at 50 inch-tons and not so good about YY at 23 inch-tons. Box piles have a better balance about both axes, but, subject to framing to catch the YY axis, broad-flange beams are the most economical steel section about the XX axis.

Timber, however, of 14-inch-square section, has an ultimate value of about 100 inch-tons, which is better than any other reasonable alternative, and it is of course equally strong about both axes. A single timber pile would not have the K.E. value of 400 inch-tons said to be necessary, but it would be twice as strong as the existing pile had been about XX and nearly five times about YY. All these figures apply to blows at low water. Theoretically the K.E. capacity should be the same for blows at any point, (for example, well above high water), but as the point of application gets nearer to the support, the value of the blow increases and to offset this in design No. 19, additional resilience is provided by the bonded-rubber units.

Although design No. 19 does not provide the full K.E. value of 400 inch-tons originally aimed at, it can be constructed for at least half the cost of No. 18; it is, moreover, simple and direct to build and, most important of all, can be readily repaired when in use. For these reasons No. 19 was adopted as the repair replacement for No. 17, in preference to No. 18.

No. 20 (*Figs 23*).—This design is an advance preliminary proposal for the use of Neidhart units. These units use rubber in a rolling compression manner, the principle having been designed and patented by a Swiss engineer named M. Neidhart. A series of links and a " lazy " unit are necessary, as used in design No. 20, to develop resilience in two directions, and until some full-scale trials have been made it cannot be stated whether or not this particular usage is too intricate for maritime work. The principle of the unit is, however, most ingenious; it is more positive than bonded rubber and if a simple mechanism can be evolved to go with it, it should be possible to apply it in various ways for jetty fendering. Brief calculations

Figs 23



DESIGN NO. 20

based on information supplied by the Andre Rubber Co. are given in the Appendix.

Comments

Vulnerability of Fendering

The fundamental equation for kinetic energy is:

$$K.E. = \frac{1}{2} \frac{WV^2}{g}$$

So far as jetty fendering is concerned this means that the energy to be dealt with is directly proportional to the weight of vessel involved and to the square of its velocity of approach.

A useful approximation to remember is that 1,000 tons moving at a velocity of $\frac{1}{2}$ foot per second has a K.E. value of 50 inch-tons. Thus, if fendering has been designed on the basis of half the weight of a 1,000-ton vessel striking it at $\frac{1}{2}$ foot per second then it should have a resilience capacity of 25 inch-tons. But in practice it may well receive a head-on blow from a 2,000-ton vessel moving at 1 foot per second, that is, applying a K.E. of 400 inch-tons. Unfortunately, the visual differences between a vessel of 1,000 tons and one of 2,000 tons and between speeds of $\frac{1}{2}$ and 1 foot per second respectively are not very striking, but the effect of one is sixteen times greater than the other.

It is almost certain that, no matter how well a fender has been designed and however well it may stand up to everyday usage, it will one day receive a blow that will destroy it. And the range between normal usage and a not unreasonable accidental possibility is so high that it is not really practical to cater for the latter. This is more especially true of smaller fenders in which the day-to-day usage does not warrant a K.E. value much in excess of 100 inch-tons. When larger vessels of 20,000 tons or more are involved and fender capacities of 1,000 inch-tons are justified, the possible upper limit of accidental blows is proportionally much lower since far greater care will be exercised in the handling of large vessels. Nevertheless, even with the largest of vessels the range is likely to be greater than can be accommodated in the factor of safety of the fender device and the probability of a complete failure must be faced. Timber rubbing pieces that will be knocked off rather than stand up to heavier punishment and which can be readily spiked on again are invaluable. Whole timber fender-piles were used almost exclusively before the war when timber was cheap and plentiful, and, except for large fendering capacities of 200 inch-tons or more, timber is probably still the best material.

Berthing Velocity

The tonnage of vessels likely to use a jetty can sometimes be given within reasonable limits, but berthing speeds—which are far more significant than displacements—never can. A reasonable average design figure is

½ foot per second, although on the basis that large vessels will be handled with more care than smaller ones, an approach velocity of, say, 1 foot per second should be adopted, where practicable, for vessels of less than 1,000 tons. At each berth there should always be sufficient total resilience to cater for the whole weight of the vessel at its adopted velocity. Individual fender units should be sufficient to take half the weight of the vessel on the assumption that if, as is most likely, the vessel comes alongside so that the bows strike first and then the stern swings round and strikes a second blow, each of the two blows will be equal. When large vessels are involved and isolated fender clumps are used at wide intervals, it is a convenient arrangement to provide three such clumps each capable of taking half the calculated total K.E. In this way a 50-per-cent "factor of safety" is available.

Berthing Angle of Approach

Blows at right angles to the lines of a jetty are not too difficult to deal with and numerous types of fenders suitable for this are available. Longitudinal blows are so difficult to handle that they are usually ignored. There is no justification for this whatsoever, because longitudinal or glancing blows cause more damage than any others. When a vessel approaches a jetty broadside-on it is obviously going to be pulled up completely, and unless it is out of control the berthing master will keep the speed low and the chances are that the bows will strike first and the stern swing round to strike again. A good deal of energy must be used up during this swinging round through the water. Angular approaches, however, will be made at higher speeds and with such blows the whole weight has got to be arrested, since the vessel will hardly swing round about its shorter beam axis. The best method of dealing with glancing blows is to provide a fender that will give way to them rather than try to stand up to them. Floating fenders help in this respect or a heavy clump, as in *Figs 11*, which hangs freely at a bottom dead centre should be lively in this position and should readily move back out of harm's way. It must be admitted, however, that a really good solution has not yet been devised.

Fender Piles

Fender piles must be equally strong about both axes. When that is ensured there still remains the question of the point of application of berthing loads. When floating fenders are used there is no argument; the load is applied at water level and in large tidal ranges the bending moment induced in the fender pile at low tide may be very high indeed. If the design provides for a spring device at deck level to come into play at all states of the tides, then a very special type of fender pile may be necessary. If floating fenders are not used then where wall-sided vessels are concerned it may be argued that the effect in impact load will always be near deck level because the lower lengths of the fender pile will deflect away from any

blows, leaving the upper length in contact with the vessel. In many cases this undoubtedly occurs and the usage of some jetties may be so clearly defined as to justify adopting this as a firm basis of design. But most jetties have to be prepared to take mixed traffic, in which case some form of compromise cannot be avoided. The cost of specially made built-up girders for fender piles at, say, 10-foot centres along a jetty is never warranted and if a spring at deck level can be fully compressed at high water but not at low water, the fact just has to be accepted. It would be foolish, however, to make the spring too powerful, but conversely it should be remembered that the extra resilience of the fender pile itself at low water will help towards a balanced overall design. In this connexion it may be noted that resilience is proportional to the square of stress, and reference to Table 1 will show that the ultimate value for timber can be quite remarkable.

Costs

A steel fender-pile or one of prestressed concrete cannot be provided for much less than £200 ; a very long and heavy steel one may even cost £400. The extra cost of a bonded-rubber spring, including housing in the deck, will hardly exceed £50 and at that is always worth while. Such a combination would give a K.E. capacity of about 100 inch-tons—rather less than more—and with fender piles at 10-foot centres the cost of fendering would be between £25 and £45 per linear foot for a K.E. value of 10 inch-tons per lineal foot.

Simple gravity clumps such as in *Figs 11* are the cheapest form of really high K.E. values. Four such clumps would give 2,000 inch-tons at a cost of something less than £4,000. For the same K.E., three bonded-rubber groups as in *Figs 18* would be needed, at a cost of about £10,000. The prestressed wire springs used at Teesport [1] probably cost the same as did, possibly, the steel springs and framed steelwork used at Fawley.[2]

Apart from the gravity clumps, it seems that the cost in terms of K.E. is about £500 for 100-inch-tons, which is similar to single fender-piles. But to keep the cost of fendering per linear foot of jetty comparable, the heavy framed groups would have to be spaced at between 200- and 300-foot centres. From a berthing point of view, this is seldom acceptable. It is unlikely that it will ever be possible to fender a jetty continuously with really high-K.E.-capacity fenders. But if single gravity clumps as shown in *Figs 11* were provided at 15-foot centres, the cost might be kept to about £50 per linear foot of jetty for a K.E. value of 30 inch-tons per linear foot. Future developments will most probably be along these lines.

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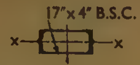
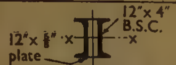
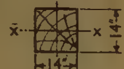
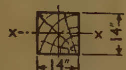
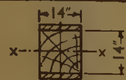
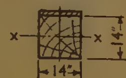
1 "Two Modern Reinforced-Concrete Jetties." *Constr. Engr*, vol. 7, No. 2, p. 28 (Feb. 1951).

2 "A Large Jetty on Southampton Water." *Conc. and Constr. Engng*, vol. XLVI, No. 9, p. 269 (Sept. 1951).

TABLE 1.—PROPERTIES OF SOME FENDER PILES

Based on a 60-foot pile, simply supported at both ends and loaded at centre with point-load W. Central deflexion δ. Limiting steel stress: 15 tons per square inch. Timber stress: 2,800 lb. per square inch. E of steel: 30×10^6 lb. per square inch. E of timber: $1{\cdot}18 \times 10^6$ lb. per square inch (Douglas Fir).

Type of pile	Cross section	Weight: lb. per foot	About XX axis				About YY axis				Energy/weight ratio	
			Z: inches³	Energy absorption: inch-tons	W: tons	Deflexion: inches	Z: inches³	Energy absorption: inch-tons	W: tons	Deflexion: inches	XX	YY
B.S.P. box pile No. 5		134	180	47	15·0	6·2	137	31	11·0	5·4	13·3	8·8
B.S.P. box pile No. 5, reinforced with two 12″ × 1″ steel plates		215	308	80	26·0	6·4	170	38	14·0	5·4	13·8	6·5
Broad flange beam 16″ × 12″		110	188	48	16·0	6·1	48	16	4·0	8·1	16·3	5·5
Broad flange beam 18″ × 12″		122	233	52	19·0	5·5	52	17	4·3	8·1	16·3	5·5
Broad flange beam 20″ × 12″		135	281	57	23·0	4·9	56	18	4·7	8·0	15·7	5·2
Broad flange beam 24″ × 12″		165	405	68	34·0	4·0	66	22	5·4	8·1	15·3	5·0

Two channels welded toe to toe		103	80	41	6·7	12·0	134	32	11·0	5·8	15·0	11·8
Two channels welded toe to toe, reinforced with two 17" × 1" plates		217	203	82	17·0	9·7	231	55	19·0	5·8	13·9	9·4
Compound section		124	157	48	13·0	7·3	67	23	5·6	8·1	14·4	6·9
Compound section		358	456	131	38·0	6·9	301	68	25·0	5·4	13·8	7·1
Compound section		436	964	176	80·0	4·4	398	80	33·0	4·8	15·3	7·0
Timber (stressed to 2,800 lb. per square inch)		46	457	23	3·2	15·0	D	I T	T	O	19·0	19·0
Timber (stressed to 5,600 lb. per square inch)		46	457	92	6·4	30·0	D	I T	T	O	76·0	76·0
Timber and steel (two 14" × 1½" steel plates)		189	7,650	72	25·0	6·0	2,950	33	9·7	6·8	14·0	6·6
Timber and steel (one 14" × 1" steel plate)		94	890	82	11·0	16·0	1,286	14	4·2	6·8	40·0	7·0

Conclusions

The design of jetty fendering can never be exact and precision must always give way to common-sense compromise. Glancing longitudinal blows are more important to guard against than blows at right angles to the cope line. Flexibility must be provided in fendering, and fender piles must be equally strong about both axes. Blows should be "ridden" not "taken." Ease of maintenance—to the extent of complete renewal—must always be borne in mind.

The Paper is accompanied by five sheets of drawings and diagrams, from which the Figures in the text have been prepared, and by the following Appendix.

APPENDIX

Bonded-Rubber Units

1. For a 45-degree direct-shear movement, the shear stress developed in the rubber may be taken as 160 lb. per square inch. A displacement of 45 degrees could not be accepted for a mounting which might be subjected to it at frequencies high enough to be measured in cycles per minute. But for a maritime fender unit which might only be used to such a limit once a month or even less frequently, it can be accepted; and a shear stress of 160 lb. per square inch will give a reasonably accurate assessment of the ultimate load that the unit will develop for that displacement. The maximum resultant kinetic-energy value of the unit is then half the product of the load and the displacement.

2. For movements less than 45 degrees, the shear stress developed is by no means proportional to strain, so that the K.E. value does not follow a straight line. Intermediate values, however, are of no particular interest in jetty design and for practical purposes it is quite sufficient to simplify the problem into an assessment of only the ultimate values.

3. Some typical bonded-rubber units and their ultimate capacities are:

(*a*) 9 inches diameter × 3 inches thick = $4\frac{1}{2}$ tons × $\frac{3}{2}$ inch = 6 inch-tons

(*b*) 14 ,, ,, × 3 ,, ,, = 11 tons × $\frac{3}{2}$ inch = 16 ,,

(*c*) 21 ,, ,, × 6 ,, ,, = 25 tons × $\frac{6}{2}$ inch = 75 ,,

Neidhart Rubber Units

4. These units contain four solid rubber cylinders. The diameters and length of the rubber can be of almost any dimension but for practical purposes it is convenient to consider only diameters of 1, 2, and 3 inches, and lengths of 6, 12, 18, and 24 inches.

5. At present two ratios of design are available and the torque capacities of a unit with four rubbers and a 45-degree displacement are:

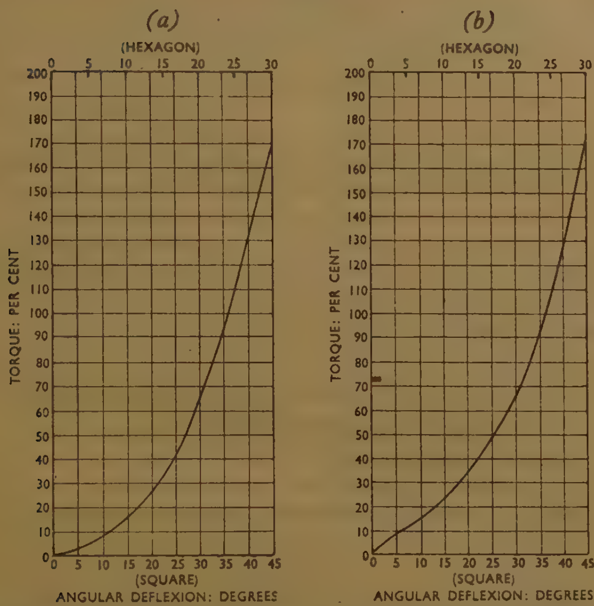
(*a*) 1,140 lb.-inches for rubbers 1 inch in diameter × 1 inch in length.
(*b*) 1,860 lb.-inches for rubbers 1 inch in diameter × 1 inch in length.

6. It would not be wise, however, to move the units to the full value of 45 degrees or 170 per cent torque. The limit should be 35 degrees or 94 per cent torque for case (*a*), and 30 degrees or 66 per cent torque for case (*b*). (See *Fig. 24* (*a*) for case (*a*) and *Fig. 24* (*b*) for case (*b*).) Torque figures are then as follows:

(*a*) $\dfrac{1,140 \times 94}{170} = 630$ lb.-inches for rubbers 1 inch in diameter × 1 inch long.

(*b*) $\dfrac{1,860 \times 66}{170} = 724$ lb.-inches for rubbers 1 inch in diameter × 1 inch long.

Figs 24



(Compiled from Test Results.)
Ratios 1 — 2 — 3
$T = 1{,}140 \times D^2 \times L$ (square)

Ratios 1 — 1·8 — 2·64.
$T = 1{,}860 \times D^2 \times L$ (square)

Graphs showing Neidhart-Unit Deflexion Characteristics

7. For other sizes of rubber, the torque is proportional to the rubber dimensions as follows:

(*a*) $T = 630 \times D^2 \times L$

(*b*) $T = 724 \times D^2 \times L$

where T denotes torque in lb.-inches at 35 degrees for (*a*), and 30 degrees for (*b*).
D denotes diameter of rubber in inches when free.
L denotes length of rubber in inches when free.

8. Torque values in tons-inches for various sized rubbers of the two ratios when at (*a*) 35 degrees, and (*b*) 30 degrees are as shown in Table 2.

Table 2.—Torque Values

Length: inches	1-inch diameter		2-inch diameter		3-inch diameter	
	a	*b*	*a*	*b*	*a*	*b*
1 . .	0·281	0·323	1·12	1·29	2·53	2·9
6 . .	1·69	1·94	6·72	7·75	15·2	17·4
12 . .	3·38	3·9	13·4	15·5	30·4	34·8
18 . .	5·06	5·85	20·2	23·2	45·6	52·2
24 . .	6·75	7·8	26·9	31·0	60·8	69·6

9. In practice two units have to be brought into operation and, for jetties, 1-inch diameter rubbers are not strong enough to bother with, and the weaker of the two ratios (*a*) can be disregarded. A reasonable range therefore comprises :

(i) Two 2-inch-diameter units, 6 inches long = 15·5 tons-inches
(2·58 tons at 6 inches arm)

(ii) " " " 12 inches long = 31 tons-inches
(3·1 tons at 10 inches arm)

(iii) " " " 18 inches long = 46·4 tons-inches
(4·64 tons at 10 inches arm)

(iv) Two 3-inch diameter units, 18 inches long = 104·4 tons-inches
(5·22 tons at 20 inches arm)

(v) " " " 25 inches long = 139·2 tons-inches
(6·96 tons at 20 inches arm)

10. Neidhart units have to be used with a link system and this can be proportioned to give whatever force and movement may be desired, subject, of course, to the product of the force and the lever arm being equal to the torque on each unit.

11. It should be noted that the torques referred to are moments and not kinetic energy.

12. The K.E. conversion factor is given by the fundamental equation :

$$\text{K.E.} = \frac{T\theta}{2}$$

where T denotes torque
θ " angular movement in radians.

13. As indicated in paragraph (6), the maximum angular movement is 30 degrees, that is, ½ radian (approx.). Therefore :

$$\text{K.E.} = \frac{T}{2} \times \tfrac{1}{2} = \frac{T}{4}$$

14. Hence the K.E. values for the units referred to in paragraph (9) are :

(i) Two 2-inch-diameter units, 6 inches long $= \frac{15 \cdot 5}{4} = 4$ inch-tons

(ii) " " " 12 inches long $= \frac{31 \cdot 0}{4} = 8$ "

(iii) " " " 18 inches long $= \frac{46 \cdot 4}{4} = 12$ "

(iv) Two 3-inch diameter units, 18 inches long $= \frac{104 \cdot 0}{4} = 26$ "

(v) " " " 24 inches long $= \frac{139 \cdot 2}{4} = 35$ "

Discussion

The Author introduced the Paper with the aid of a series of lantern slides.

Mr Harry Ridehalgh said it was a pity that the Author had not been able to conclude his Paper by describing the perfect fender ; there was, however, little doubt that what he had recorded would be of great assistance to those engaged on that difficult problem.

In spite of what the Author had said in his introductory remarks about the strength of individual piles as fenders, Mr Ridehalgh still preferred to see the piles laced together in groups, because, in his opinion, the individual pile was rather vulnerable, and when it was damaged it exposed its neighbour to further damage.

He was also doubtful about the use of rubber. He had visited one of the sites referred to by the Author and had seen the canvas cover shielding the bonded-rubber unit. Presumably the rubber did not last quite so long when exposed to the weather; could the Author give some idea of the life of a bonded-rubber unit? How would the rubber fare in a tropical climate?

Design No. 20 looked very attractive but perhaps a little mechanical, and Mr Ridehalgh would expect some trouble from the link motions. Undoubtedly the concrete gravity fenders were the best so far put forward. They were simple to construct, efficient in operation, and had a high kinetic-energy-absorption value.

It was observed that the Author had departed from the vertical suspenders and was suggesting that they should be inclined. Mr Ridehalgh had recently completed a quay in Sunderland in which inclined chains had been installed but with four-point rather than three-point suspension, and they were proving quite effective. They did not return to the rest position with such a bump because, as the ship moved away, so the fender slid down the side of the vessel. Trouble had been experienced with the connexion to the concrete block, but it was not serious. They were built to take an appreciable horizontal blow.

The Author's prophecy that sooner or later quays would be built with continuous gravity fenders all along the face was already fulfilled, because that same wharf in Sunderland to which he had referred had continuous fenders at 11-foot centres throughout its whole length.

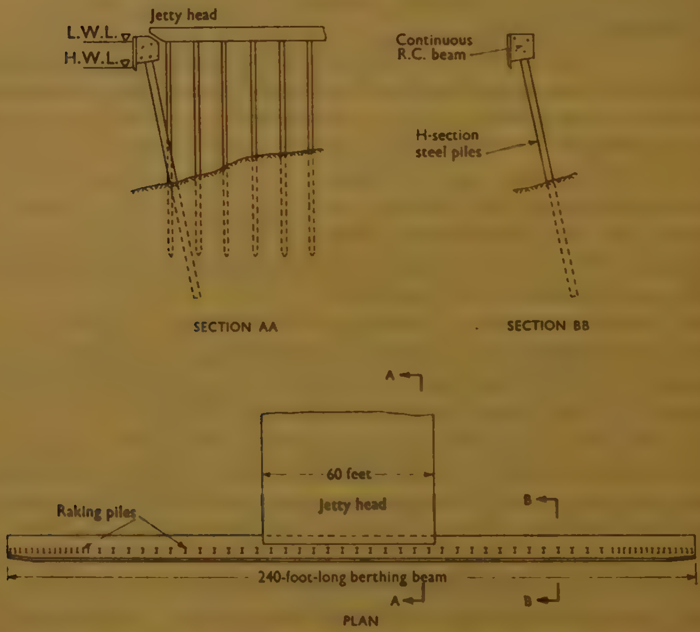
The forecast of cost given in the Paper was remarkably accurate. At the wharf already referred to, the fenders worked out at £52 10*s*. per foot of quay and had a kinetic-energy-absorption value of 37 inch-tons per foot. The great advantage of the system was that practically the whole of the quay was protected. That was very important. Mr Ridehalgh had in mind a wharf which had suffered damage at least twice yearly for the past 7 years, because the fenders were spaced too widely. In his opinion the ideal system should provide complete protection for the wharf behind. It should also be made to cater for the glancing blow and be of such a design that the exceptional blow would not damage the wharf itself. The nearest he had been able to get to that ideal was shown in *Figs 25*.

The principle involved was the prestressing of a raking steel pile by loading it at the head with a large block of concrete. As a result the kinetic-energy value of the pile as a simple cantilever was considerably increased, and could be increased still further by the use of high-tensile steel. In practice, the raking piles were in single line along the front of the wharf, and they were laced together at the top by the very heavy and stiff reinforced-concrete beam, so that no matter where the beam was struck, all the piles were brought into operation simultaneously. The worst case arose when a blow was delivered at or near the end, in which case the other end would move outwards. To cater for that the end piles

had been placed closer together and they were stressed in the forward direction so that, in fact, they could move still further forward. Such an arrangement could be designed to have almost any desired energy-absorbing capacity, and there was no difficulty in designing a berthing beam to deal with vessels up to 44,000 tons. The piles were, of course, quite capable of catering for an appreciable glancing blow.

Other advantages included complete protection of the wharf behind.

Figs 25



Continuous Berthing Beam on Raking Steel Piles

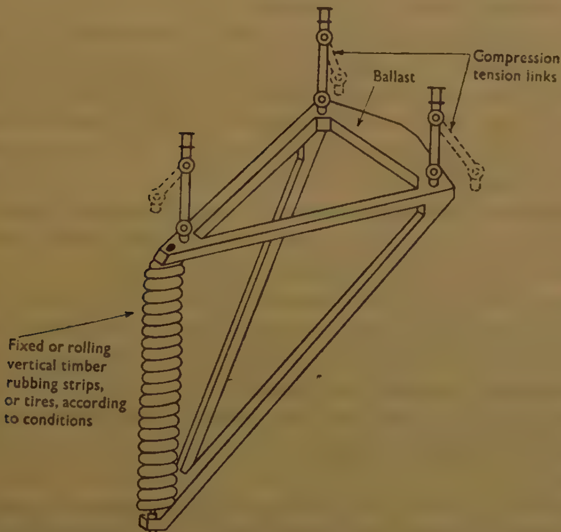
In addition, the berthing face offered to vessels coming alongside was identical with that of the normal dock wall, and its depth could be varied to cover wide tidal limits, which was often one of the difficulties when designing fenders. The piles raked away from any vessel lying alongside so there was no danger of fouling the piles when the vessel took a list as so often happened. One other advantage was that the berthing beam could be built along the front of any existing openwork jetty without having to strengthen that jetty. That afforded an appreciable saving, not only for an existing jetty, but for new jetties also, for the structure behind had not to be designed to take ship blow.

Mr Ridehalgh's firm had recently designed four wharfs of that type

and construction was about to commence. It was hoped that later on he would be in a position to give some account of their operation.

Professor A. L. L. Baker said that the development and research described in the Paper were of the kind which could be done only on full scale and with a variety of berthing conditions. The Author had given much useful information and had established that the values of kinetic energy which had been suggested from time to time within the past 10 years were very near the mark. The performance of the various kinds of fenders confirmed that.

Fig. 26



THREE-POINT-SUSPENSION FENDER IN REINFORCED OR PRESTRESSED CONCRETE

The new development of the use of rubber on top of the fender was very promising. It provided a reasonable kinetic-energy absorption for certain cases up to about 100 inch-tons, but for kinetic-energy values of more than 100 inch-tons either a number of rubber-cushion fenders had to be grouped together or some kind of gravity fender used. A significant point was that rubber-cushion fenders could develop only a small movement in comparison with gravity fenders and were, therefore, necessarily subject to greater impact forces and therefore greater wear and tear. They were, however, much lighter.

The Author had encountered the problem of the glancing blow, and Professor Baker made two suggestions for dealing with it. *Fig. 26* showed a framework suspended from three points with links which could take compression as well as tension, with joints rather like the joints at the

base of a ship's derrick. The three points at the top were in the deck and the fender had the advantage of avoiding low-water bracing. The fender itself was made either as a reinforced-concrete rolling-pin or, if continuous with the frame, of circular section and covered with old tires, which would rotate slightly and ease the glancing blows. The fender itself would also move freely longitudinally as well as inwardly and present only a small area to wave slap. That would appear to be a logical form for a fender and perhaps a simple way of getting high shock-absorption through large movement. The weight need not be excessive, the whole fender could be easily suspended, and the face could be easily replaced in case of severe damage.

Figs 27 illustrated a model-experiment on the vertical type of gravity fender—which was generally necessary where there was a big tidal range. It had splayed links at the top and bottom and splayed corners to rubbing strips. *Fig. 27* (*a*) showed how an approaching ship, moving longitudinally, partly rotated the fender and pushed it sideways until it disengaged. The fender thus evaded heavy oblique impact. *Fig. 27* (*b*) showed how the fender had dodged the blow instead of taking it. It had receded under the inward pressure and was about to press the ship out again. Such a fender should be subject to very little wear and tear from the glancing blow on sites where such blows commonly occurred.

Fenders of that type had been put up as a temporary measure at Inchkeith during the recent war. The links were splayed in the same way, and there were a pair of links at the bottom. They had worked very well during the war in protecting ammunition ships as they rounded the end of a concrete mole. Apparently prevailing winds had tended to press the ammunition ships hard against the end of the mole as they turned into the harbour.

When discussing costs, it did not seem appropriate to consider the cost per square foot of jetty. The cost should be computed on the cargo handled. That was an extremely important item, particularly for those in commercial work and oil company work. The experience gained at the Kuwait Oil Company's jetty showed that the fenders, although expensive, had been justified by their performance in allowing tankers to berth in all weathers and without delays, which could be very expensive. Another item of cost to be considered was that in all fenders in which wear and tear took place, there was always a corresponding wear and tear on the ships and whether one was responsible for a harbour or for a ship, it was most important in present times to study the ship as well as the harbour.

Mr J. H. Jellett observed that the Author had traced the evolution of flexible fenders, starting from the premise that it was now necessary to protect the jetty from the ship rather than the other way round. Mr Jellett's own most recent experience had developed on the basis of protecting the ship.

Until recently it had been possible to avoid the problem of having t

protect the ship from the jetty by assuming it to be a marine risk to be settled between shipowners and underwriters. However, the recent arrival in Southampton of the new American ship *United States* had presented an opportunity of producing flexible or compressible floating catamarans. Those had had to be constructed in any case, as spacers to hold the ship a given distance away from the quay, but had never, so far, been provided with any form of compression element. Although hampered by the usual lack of time in which to work out a design, catamarans had been produced, of the form shown in *Fig. 28.* They were 36 feet long, 15 feet wide, and 10 feet deep. They were manufactured in three pieces, separated by a large number of stacks of tires stuffed with coils of rubber hose. (The function of the hose was not to provide additional compressibility but rather to assist the recovery of the tires after compression.) Experiments showed that the chief value of a tire in such a position was not in the resilience of the tire itself but in the fact that the water contained in the centre of the tire was throttled in its attempt to escape under compression.

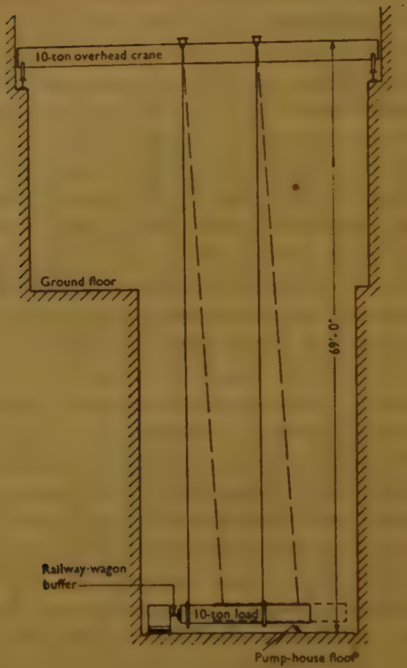
Fig. 29 was an end view of one of the catamarans. The stacks of tires could be seen and the method by which the parts of the catamaran were held together by diagonal wires. Longitudinal or rubbing stresses on the face were taken by large stout timber stubs built into the centre portion and acting as sliding spigots in rectangular sockets in the outside portions.

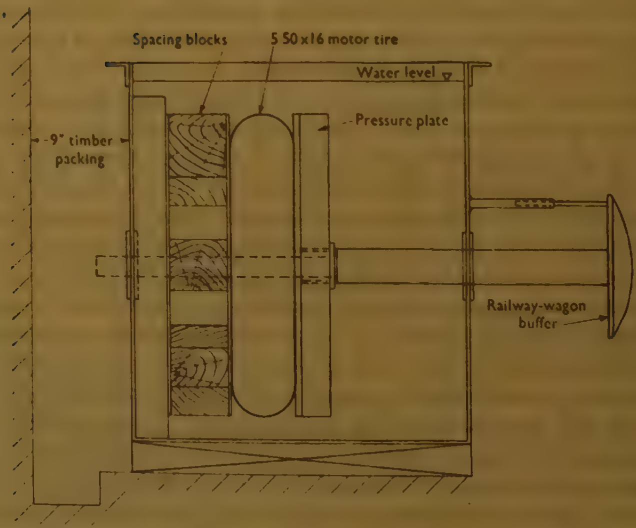
Fig. 30 showed the centre section with the stacks of tires in position. If any set of tires happened to require repairs it could be drawn up and re-slung from staples. The fenders floated very deep, with a freeboard of little more than 1 foot. Up to date they had been very successful.

There had been little information available about the compression strength of a tire, so an experiment had been carried out in the pump-house at King George VI Dock at Southampton. A 10-ton double pendulum in the pump pit was slung from an overhead travelling crane. It was arranged, as shown in *Figs 31*, to swing against a railway-wagon buffer, the stem of which passed into a tank full of water housing the tire under test. The weight of the blow could be adjusted by the extent to which the pendulum was drawn back. A dead-set plunger measured the extent of the compression, but insufficient information was gained from this and in the end it was found necessary to photograph the impact with a high-speed cinema camera making 150 exposures per second, and to analyse the movement from the pictures. The maximum compression obtained was equivalent to only 25 per cent of that allowed in the movement of the fender itself. A straight-line diagram from that would give the possible figure of 300 tons compressive value of the catamaran under the full compression of 15 inches, but all the indications were that the straight-line diagram gave an under-valuation. In other words, when compression increased other factors came into play, so that under full compression probably the load on the fender would be much more than 300 tons.

Mr R. A. Stephenson said that his problem also was to protect the ship

Figs 31





PENDULUM EXPERIMENT TO DETERMINE ENERGY ABSORPTION OF TIRE UNDER WATER

Figs 27

(*a*) (*b*)

MODEL-EXPERIMENT ON A VERTICAL TYPE OF GRAVITY FENDER

Fig. 28

FRONT VIEW—COMPRESSIBLE CATAMARAN OR " DUMMY " *

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* Reproduced by courtesy of the Docks and Inland Waterways Executive.

Fig. 29

END VIEW—COMPRESSIBLE CATAMARAN OR "DUMMY" *

Fig. 30

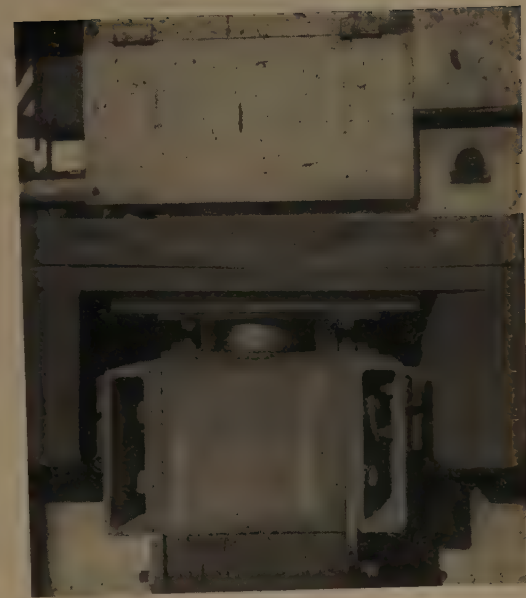
CENTRE SECTION—COMPRESSIBLE CATAMARAN OR "DUMMY" *

* Reproduced by courtesy of the Docks and Inland Waterways Executive

Fig. 33

JETTY AT DAGENHAM DOCK FOR COAL AND OTHER BULK CARGOES

Fig. 34



BONDED-RUBBER-IN-SHEAR UNIT ON TOP OF FENDER PILE, WITH SIDE-SNUBBERS FITTED LATER

Fig. 35

BUSH-TYPE RUBBER MOUNTING (SEE ALSO *Figs 36*) ON RIGHT, BONDED-RUBBER TYPE ON LEFT

Fig. 37

RUBBER COMPRESSION CYLINDERS AROUND HEAD OF FENDER PILE

from the wall. He had not actually constructed any resilient fenders for that purpose, but had spent much time looking into what could be done. One of the problems was to discover what the ship could stand under given conditions of loading, and it would be interesting to know whether the Author had any information available on that point. Also, had he any experience of the effect of spacing on the vulnerability of isolated fender units taking vessels ranging from tugs to large liners?

Design No. 1, described in the Paper, used rubber in compression and the Author had suggested, by implication, that the energy-absorption followed what was approximately a straight-line law. Mr Stephenson believed, however, that rubber in compression gave something much less favourable than a straight line and that the energy-absorption was about one-fifth of the load times the displacement at the working load. That inefficiency was one of the reasons why, in the Liverpool docks, the idea of using rubber in compression had been abandoned and consideration had been given to rubber in shear—but it was felt that capacities would not be high enough for the job in mind.

In design No. 9, the vertical strips were cantilevered above and below the concrete body of the fender, and Mr Stephenson wondered whether there was any evidence of damage to them which might suggest that the fender had been pushed hard against the stops.

In design No. 15, where four pairs of buffer units were framed to work together, had the Author been able to ensure an equal distribution of the load among them?

In most of the Author's designs, the rubbing strips were disposed vertically, but it seemed to Mr Stephenson that strips placed horizontally would stand a better chance of not getting torn off. He also wondered whether the characteristics of the timber were important; for example, was greenheart better than elm?

Had any instance been recorded of the upward movement of a gravity fender being arrested by its riding up beneath the flare of a ship's bow? That would almost certainly smash the links of the fender.

When considering the question of what impact energy should be allowed for, Mr Stephenson had concluded that the generally accepted value of half the theoretical kinetic energy of the ship was a good design figure. It had been calculated that if there were a purely rectangular ship which, when approaching, landed on one corner, the energy absorbed by that impact would be about 25 per cent of the total energy of the ship. The ship would then continue to swing with 75 per cent of the original total energy. Since ships were not rectangular and the blow was not received at the extreme end, the proportion of the energy absorbed would rise, probably to about 40 per cent for a shoulder blow on an average ship. In addition, the effect of a wind on the side of the ship had to be allowed for; the reaction to the wind force had to be subtracted from the resistance of the fender, whose effective energy-absorbing capacity was correspondingly

reduced. Summing up, the figure of 50 per cent seemed to be justified both theoretically and practically.

Mr B. F. Saurin stated that the Author, when discussing the vulnerability of fenders and berthing velocity, had pointed out that ships normally came alongside and struck one end first, and had suggested that the energy to be used in calculating the capacity of the fender was about one-half the total energy of the ship moving uniformly at impact velocity. That recommendation required special consideration in connexion with the berthing of oil tankers. Tankers were sometimes berthed against a continuous wharf or quay, as in the case of the Kuwait jetty, but a much more common form of berth for a tanker was a pair of dolphins with an operating platform in between. Those two dolphins were spaced no farther apart than about one-quarter or one-third of a ship's length. It would seem that if a ship struck such a dolphin near its centre, it would be much harder on the fendering than if it struck the quay at or near its stem. That general impression was confirmed by the following calculation.

If the ship were to be represented as a solid bar of length L feet, weight W, and radius of gyration k, with its centre of gravity G at the mid-point; and if it were to move so that some point X, distant a feet from G, was about to strike a fender with impact velocity V feet per second, then it could easily be calculated that the amount of energy which would be transmitted to the fender by the impact was given by the following expression:

$$\frac{Wv^2}{2g}\left(\frac{1}{1+\frac{a^2}{k^2}}\right)$$

Obviously the energy transmitted was in inverse proportion the value of a, and if a were taken as equal to $\frac{L}{2}$ the amount of the energy transmitted would be theoretically one-quarter of $\frac{Wv^2}{2g}$.

If, on the other hand, a were zero, the fender would have to absorb the whole of the energy. It was not difficult to indicate by a curve how the energy transmitted varied with the distance from the centre. If the contact point X was one-third of the length of the ship from the bow or stern, corresponding to one of the dolphins against which the tankers were berthed, the proportion of the total energy which was transmitted to the fender was no less than three-quarters of $\frac{Wv^2}{2g}$. Thus in the case of the continuous wharf it was possible to use a figure much less than that recommended by the Author, namely, only a quarter of $\frac{Wv^2}{2g}$. If, however, it was an ordinary tanker berth consisting of two dolphins, it was necessary to cater for three times that much energy.

Professor Baker had found as a result of his observations at the Heysham jetty that 40 per cent of $\frac{Wv^2}{2g}$ was absorbed in the bell-type gravity fenders. The energy curve would show about 50 per cent in that particular case, but presumably the discrepancy was taken up in the ship's structure and movement in the water.

Probably very little energy was actually absorbed in the ship's hull where gravity fenders were used. With stiffer fenders it was said that much more was absorbed. The calculations were not a final guide, and Mr Saurin suggested that Professor Baker had indicated the right way to tackle the problem, namely, by taking observations on existing fenders. Gravity fenders afforded a ready means of measuring energy absorption.

Mr John Cuerel said that there had been much discussion on the use of rubber—mainly in shear—and on the basis of figures given in the Paper the absorption was only 5 foot-tons per cubic foot with the bonded system, and a little less than 4 foot-tons per cubic foot with the Neidhart system. He suggested that the proper way to use rubber was in compression, when the absorption could be as much as 12 foot-tons per cubic foot of rubber, and the application of the load was very much easier.

The Author had given some costs of gravity fenders, from which it would appear that in 1944 a 425-inch-ton job could be built for about £700. The energy-absorption given was at low water and it would of course be less at high water—probably about 325 inch-tons. Therefore, the cost would be about £2 per inch-ton. Since 1944 the cost had risen and Mr Cuerel suggested that it might now be about £3 per inch-ton, which gave a figure of something like £6,500 for a 2,000-inch-ton fender.

The economics of gravity-fender systems were not confined to the fenders themselves. The weight of such a fender might be of the order of 150 tons so that it would require three more piles, which, in unfavourable circumstances, might lead to another £1,500, making a total of £8,000.

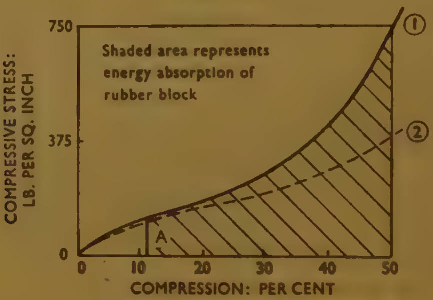
Some jetties had recently been built which used compression-rubber fenders, arranged in groups. Each group was capable of absorbing 75 foot-tons (900 inch-tons). The cost was about £1,900, which was equivalent to £4,200 for a 2,000-inch-ton fender, compared with the £8,000 cost of the gravity fender. The cost of such compression-rubber fenders included the rubber blocks, which had been purchased when rubber was 5*s*. per lb. (it was at present down to 2*s*. per lb.); it included all fittings and galvanizing thereof, welded and zinc-sprayed steel box-girder, timber sheathing, and fixing complete. Therefore, it would appear to be fair to say that at the present time compression-rubber fenders could be designed at half the cost of the gravity fender.

Of course, compression rubber had many other advantages over the gravity fender by way of lessened vulnerability, the ease with which it could duck longitudinal blows, the avoidance of up and down rubbing against the ship, and generally lessened maintenance and ease of repair.

It was now possible to obtain rubber blocks of sizes up to 5½ cubic feet in volume (20 inches diameter by 30 inches long), and each block was capable of absorbing 60 foot-tons of energy. With a unit employing three such blocks there was a potential of about 180 foot-tons, and the unit could be very easily and cheaply made. When it was considered that 180 foot-tons was the capacity of two of the huge gravity units such as were used on the Kuwait jetty, the relative advantages of rubber were obvious.

It had been said that the compression curve for rubber was not so good as it might be for absorbing energy. Mr Cuerel had made a number of trials and suggested that that was not the case. A typical compression curve was shown in *Fig. 32*. In order to avoid "wobble" due to waves

Fig. 32



TYPICAL COMPRESSION CURVE

Curve 1 taken on basis of undeformed rubber block.
Curve 2 taken on basis of deformed rubber block (actual average stress).

the rubber was given some precompression to A. The curve rose steeply at the higher pressures but the stresses had been measured on the basis of an undeformed rubber block. When compressed, the diameter had increased so that the true stress had been less, as indicated on the diagram.

Mr N. N. B. Ordman agreed with the Author that the design of jetties and fenders, particularly fenders, could never be exact; but there were some aspects of design in which a certain exactness was required. For instance, there was a need for more information than was yet available regarding the approach speeds which could be expected. Although he agreed with the Author that very high accidental speeds could never be used as a basis of design, nevertheless it was true that the more data which was available regarding approaching speeds, the more accurate would be the knowledge of what was normal, and it was that which was normal which should be used as a basis of design, together with some arbitrary factor of safety based on experience.

Considering the seriousness of the problem, the amount of published information based on observation seemed to be remarkably small, and he

had been able to find only three or four published records of results of observations.

In the section on berthing velocities, the Author had suggested a design speed of $\frac{1}{2}$ foot per second generally and 1 foot per second for vessels of less than 1,000 tons displacement. It would be interesting to know if those suggested speeds were in fact based on observations and if the results of such observations could be made generally available. It would also be interesting to know whether those speeds were "ahead" speeds of the vessel or velocities at right angles to the face of the jetty.

Dealing with a point which had already been raised, namely, the percentage of the impact which could be considered to be absorbed by the jetty, he had seen two published figures. Mr R. R. Minikin[1] had stated that 73 per cent of the total kinetic energy of the impact would be absorbed by the vessel, and Mr P. Garde-Hansen[2] had suggested a figure of 80 per cent. Mr Ordman had assumed that those very high percentages included the energy absorbed in causing the vessel to shear off and were therefore comparable with the figure of 50 per cent which had been quoted by the Author. Of course, a certain percentage would be absorbed in distorting the plates and the structure of the ship generally, and also in displacing large volumes of water. It might well be that that figure was the reason for the large difference between the 50 per cent quoted by the Author and the 70 and 80 per cent which Mr Ordman had seen quoted elsewhere. The figure published by Mr Minikin of 73 per cent was stated to have been based on experiments with models, and it would be interesting to know whether the Author had any knowledge of model experiments.

Another aspect of the problem of impacts which did not appear to have been dealt with so far, and upon which the Author's opinion would be valued, concerned high-intensity loading of short duration. Mr Ordman had no personal knowledge of any research which might have been carried out on the question of high-intensity loading of short duration in connexion with civil engineering structures, although it seemed probable that such research had been done concerning blast and shock waves. If such research had been carried out it would probably have some bearing on the design of jetties, if not directly on the design of fenders.

To illustrate the point he referred to p. 52 of the Paper, where it was stated that the joist "with a span of 13 feet and having both ends fixed would carry a central load of 12·55 tons at a stress of 15 tons per square inch and a deflexion of 0·29 inch. . . ." That was only true in the case of impact if 12·55 tons was the maximum force and if that force were in fact reduced to zero by resistance of the joist in bending—in other words, if a vessel were brought to rest during the deflexion of the joist. In that particular case

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[1] R. R. Minikin, "Fenders and Jetties." *Dock Harb. Author.*, vol. 28, No. 320, p. 48 (June 1947).

[2] P. Garde-Hansen, "Impact Stresses in Jetties, Wharves and Similar Structures." *Dock Harb. Author.*, vol. 26, No. 299, p. 119 (Sept. 1945).

the 15 tons per square inch was the maximum stress and the average stress was only 7·5 tons per square inch. From the figures given in the Paper it could be calculated that the duration of the bending was $\frac{1}{8}$ second. Therefore the period during which the joist suffered an intensity of stress of 7·5 tons per square inch or more was only $\frac{1}{16}$ second.

What, then, was the effect of that very short duration on the permissible stress which could be used in design? It was known that duration of stress had considerable effect on the behaviour of timber and might also affect the behaviour of other materials. It might well be that permissible stress might not be increased but might in fact have to be decreased.

Mr R. B. Kirwan said that the Paper had given him the opportunity of reporting progress and experience over the past 2 years with regard to a comparison between the Admiralty-type bonded-rubber units and certain other rubber units, on a smaller scale than had so far been mentioned, in the comparatively sheltered waters of the Thames at Dagenham Dock, where the tidal range was what the Author had described as very high!

One minor but annoying blemish in the Paper was the use of the word "clump." According to the dictionary a clump was a group or cluster, usually of trees or flowers. In the Paper, the word was used first in connexion with a group of piles and later on in connexion with a single hanging weight. Another name for the latter would make the Paper easier to read.

Fig. 33 * showed a jetty at Dagenham Dock consisting of two separate structures completed in 1939 and 1949, for the rapid discharge of coal and sugar and other bulk commodities. Although it was not very large, it took ten ships a week of displacements from 5,000 to 11,000 tons. The first part had been built before the war and was a rigid braced structure with no flexibility except for the floating fender. After the war it had been necessary to extend it to double its length in order to take a second ship. The extension had been made narrower with raking piles, and flexibility had been required to relieve both the jetty and the timber fender-piles, which spanned 44 feet vertically.

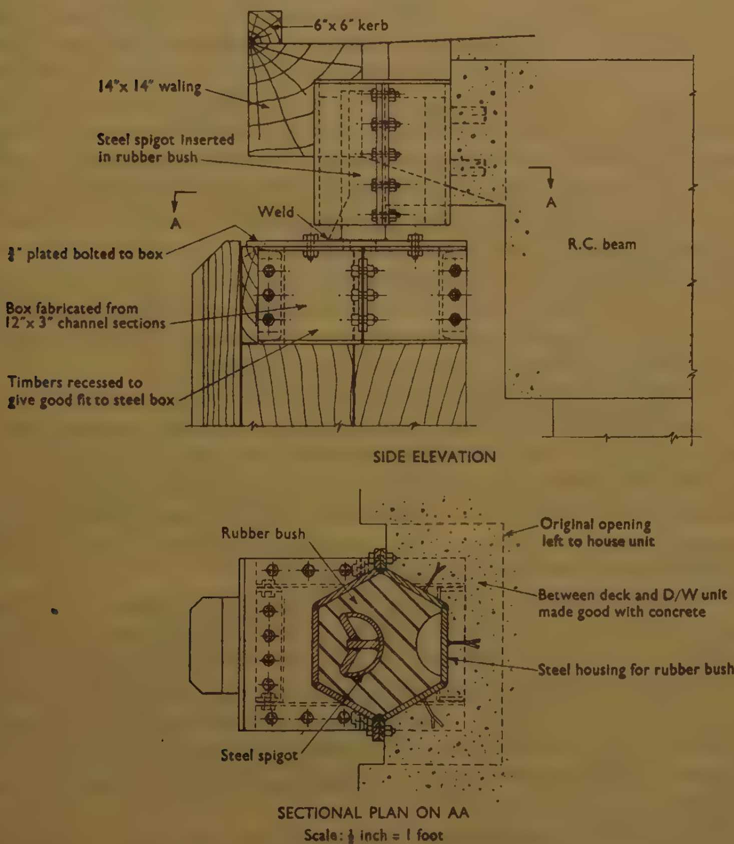
The Admiralty had kindly allowed Mr Kirwan's firm to try the bonded-rubber units and two had been fitted which worked satisfactorily, except that one had been twisted off. Side-snubbers had then been fitted (see *Fig. 34*), and also floating fenders.

Not being inclined to place all his eggs in one basket, Mr Kirwan had approached another rubber firm who had, with him, developed the unit shown in *Figs 35* and *36*. A bush of rubber was fixed to the deck inside a hexagonal steel case and enclosed a steel spigot rising from the fender pile. That rubber allowed $4\frac{1}{2}$ inches movement and absorbed 20 inch-tons under a normal blow, and it allowed some sideways movement and some rotation with good restoring action. Difficulties over development and making the prototype in time for the jetty, and also the cost of fabricating

* *Figs 33–35* are all photographs and appear between pp. 88 and 89.

small numbers, had alarmed him so much that efforts had been made to find something more simple. The result was shown in *Fig. 37* (p. 89). A 12-inch-diameter cylinder of rubber in compression behind the fender could absorb 33 inch-tons, deflecting 6 inches under a normal load of 16 tons, while a 6-inch cylinder each side could absorb 5 inch-tons from sideways blows. Although it looked crude it worked well and had a minimum of

Figs 36



BUSH-TYPE RUBBER MOUNTING

metal parts. The attempt to control torsion of the fender pile by leaving " horns " of timber projecting at the back had been a failure. The horns had split off, so the head of the pile had been cut off square and further small 3-inch cylinders were being fitted at each side. All the cylinders were $7\frac{1}{2}$ inches deep. The fenders had worked quite satisfactorily over the past 2 years, except for minor troubles, and during that time they had taken

500 berthings. Ships came in mostly without tugs, day and night, whether there was a wind or not.

The jetty was designed to take 40 per cent of the kinetic energy of ships displacing 6,000 tons, travelling at ½ foot per second. The ships had doubled in size over the past 3 years, but the difficulty had been surmounted by telling the captains to divide their speeds by the square root of two!

Mr Kirwan's objections to the bonded unit were that everything depended on the bond, which could be undermined and destroyed by rust, and to prevent rust the ordinary methods of galvanizing could not be used because bonding could not work over galvanizing. For rust-proofing one had to depend on a thin coating of rubber all over the metal—even lining the bolt-holes. When the bond or the rubber failed all was lost, whereas in compression there was usually something left between the fender and the jetty. Torsion was a proved weakness and fatigue a possible one. The force-deflexion curve for rubber in shear was not so good as that for compression, which gave a good characteristic. He therefore preferred the compression types and was particularly interested in the tubular type. It could be produced cheaply in long lengths and hung horizontally, vertically, or diagonally with wire ropes. The tubes could be hung or woven over each other to make a thick flexible screen.

The history of the tubular-type rubber fender was interesting. He had asked one English manufacturer to make some a year ago, but they had not been encouraging. Then during the summer visit of the Maritime Division to Rotterdam, some of the tubular units had been seen installed, and the makers, when approached, had said that they had been using them for 16 years in the United States. The moral seemed to be: "Join the Maritime Division and see the fenders of the world."

To quote the cost of fendering in pounds sterling per inch-ton of energy capacity, as the Author had done on p. 77, was dangerous unless the resulting thrust on the jetty was also quoted. Thus one might rough out a design for a jetty and rubber-cushioned fenders and find the cost per unit of energy capacity too high. By then choosing a harder rubber compound, the energy capacity could easily be doubled and the unit cost halved, but the thrust on the jetty would be increased and the jetty structure would cost more.

Owing to the manufacturers' difficulty in making large rubber units it became essential to consider methods of grouping several moderate-sized rubber units together to give greater capacity. The Author had stated on p. 68 that the eight bonded-rubber units in design No. 15 were framed together so that the group would act as one. That would be so if the impact blow were directed through the centre of gravity, but ships being what they were, the blow would usually be delivered in some other direction. Thus a normal blow on the outside pile of the group would find the group only about 60-per-cent efficient.

Mr Kirwan's firm had been working on a scheme to group rubber cylinders in a triangular slot in the jetty deck and transmit the blow to them via a wedge-shaped head fixed to one or more fender piles. The cylinders acted in compression and also rolled slightly. The wedge shape produced a magnification of the compression of the rubber. Thus, with a 90-degree wedge, the fender pile would move 1·4 inch for each 1 inch compression of the rubber. At the same time, good support was given against glancing blows and a certain degree of rotation was permitted.

Mr G. W. Rooke referred to the Author's conclusions at the end of the Paper, where he had stated : " Glancing longitudinal blows are more important to guard against than blows at right angles to the cope line." He (Mr Rooke) had attempted, on a previous occasion, to stress the importance of complete continuity of protection along the entire length of berthage where such berthage lay in one continuous vertical plane. He was here not referring to cases where the structure proper was set well back from a number of berthing heads set at wide intervals.

Mr Rooke was pleased to find that the importance of that requirement was now appreciated, in some quarters at any rate, and that, furthermore, a previous speaker had described a very ingenious proposal for providing complete continuity.

In his view the principle of providing units of high shock-absorbing capacity, even at such close intervals as 10 feet, was wrong, since it involved considerable expenditure in giving protection to a mere 10 per cent of the structure and left the remaining 90 per cent quite unprotected. It was essential, he considered, either to provide a fender system in itself completely continuous, or to ensure continuity by providing catamarans along the entire length of the berthage.

With regard to Mr Ridehalgh's proposal, which certainly satisfied the requirements of continuity, Mr Rooke thought that there was a weakness under conditions where the length of the berthage was considerable and sufficient to accommodate a number of ships at the same time. In such a case the effect of continuity, where a ship struck the fendering a glancing blow, would be to tend to divert her along the face of the fendering with the consequent risk that she would not be brought to rest until she had struck the bow or stern of a ship at an adjacent berth. That was certainly one way of bringing her to rest, but it could hardly be said to solve the problem in an acceptable manner. Mr Rooke did not profess to know how to combine continuity of protection with shock-absorbing capacity in a longitudinal direction.

He had suggested on a previous occasion[1] that too much money should not be spent on the provision of fenders which could, without damage to themselves, absorb the heavy blows, especially if continuity were not going to be provided. There was much to be said for taking the view that the

[1] G. W. Rooke, " Improvements in Jetty Design, with Special Reference to Systems of Fendering." Maritime Paper No. 14, Instn Civ. Engrs, 1950.

fender was there to protect the jetty at all costs; destruction of the fender was not relatively important so long as the berth structure was protected against damage.

A glance at the beginning of the Paper brought back some old memories of the brushwood fenders which had been used during the war. They had originally been used at Southampton in the late 1920s to protect certain Transatlantic liners from damage when berthing. His recollection was that the Owners of the then crack German vessels, the *Bremen* and *Europa*, had been very nervous about possible damage to those ships when berthing, so very large brushwood fenders (6 feet in diameter) had had to be provided as described by the Author. (No such nervousness had been shown by the Owners of British-built ships.) After a year or two those fenders had seemed to fall into disuse and had eventually disappeared.

With regard to the cost of fendering systems, a brushwood fender, 2 feet 6 inches in diameter by about 20 feet long, cost £26 during the war, which was 26 shillings per foot of fender. Admittedly they could not be considered for the protection of a piled structure supporting cranes, where the piles could not be allowed appreciably to deflect, but in the case of a gravity wall it was perhaps interesting to note that if brushwood fenders of 2 feet 6 inches diameter were provided at intervals of 5 feet, thus providing some measure of continuous protection, the cost would, at say £40 each at present-day costs, be £8 per foot run of wall. Although he did not know the cost of some of the other systems, it was probably many times greater than £8 per foot run of wall and how many times were these elaborate and costly devices going to be brought into action? Why not, in the case of a gravity wall, let the ship come alongside and, in the process of being brought to rest, damage to destruction say half a dozen brushwood fenders? That would entail a loss of something less than £300, but if nothing else was damaged that would be a fairly cheap reckoning.

He agreed with the Author that the brushwood fender was to some extent an emergency measure, but some of the more recent proposals were getting so expensive that it was worth while considering whether cheaper methods of protection could not be used more often, and the principle accepted that damage to fendering was permissible in an emergency in order to protect the structure, and also the ship. He had not attempted to develop a continuous system using brushwood fenders, but he put it forward as a suggestion worthy of serious consideration.

Mr Rooke had seen a very ingenious shock-absorbing device when visiting Ireland a short time ago. A berth was used for accommodating L.S.T. craft of 3,000 tons which were in service ferrying vehicles across the Irish Sea. Such craft were very weak against broadside berthing, having been designed mainly for beaching. The system adopted was very simple and effective, and consisted of steel box-piles at close intervals. The cope beam of the quay was deep, and a steel bar was passed through the box-piles just

below the bottom of the beam to support about a dozen motor tires which were threaded down, one on top of another, over the heads of the piles. Those provided the shock-absorbing capacity by means of compression between hull and pile, and also between pile and cope beam.

Mr Rooke's advice on that problem, therefore, tended towards not spending too much money on elaborate and expensive devices, but rather to make do with the simpler types of shock-absorbers.

Mr C. W. N. McGowan reminded the Author that during the discussion on Maritime Paper No. 19[1] he (the Author) had referred[2] to that discussion as " the funeral service " of the Baker-type gravity fender. Bearing that remark in mind, Mr McGowan showed a lantern-slide of one of the forty-four groups of large gravity fenders round the Kuwait pier, and three further slides illustrating the construction, disposition, action, and appearance of those fenders (which had been described in detail in the above-mentioned Maritime Paper). At the time of writing that Paper, the fenders had been in action for about 2 years, with an incidence of damage up to that time of no more than about one in 480 berthings for oil tankers of displacement up to 30,000 tons. Between that time and the present no further news had been received of any damage occurring to those fenders. The Author of the present Paper, therefore, could hardly persist in his view that the Maritime Division meeting on the 15th January, 1952, was the " funeral service " of the Baker fender in question. Those fenders were, on the contrary, behaving very well indeed.

While reminding members of some of the design details of the Kuwait gravity fenders, Mr McGowan pointed out that the designer had realized there was room for improvement in that fender and he would have preferred the concrete to have been put into the cylinders in the form of precast slabs rather than in mass concrete form. He also realized that when damage occurred there was a considerable problem in the handling of a 21-foot-long-by-6-foot-diameter cylinder weighing 43 tons. When further fenders of that type were to be installed, concrete or some other suitable material would be put in in such a way that it could be easily removed.

There was also the question of longitudinal blows of approaching ships, and the design would incorporate greater lateral strength in the supporting fender bents. Plans had been produced embodying those amendments if and when the time arrived to construct further fenders of that nature at Kuwait.

When it was found that, of the twenty examples which the Author had quoted, six were complete failures, five were a success, and with regard to the other nine the Author remained almost silent on their performance, Mr McGowan commended for further consideration the question whether

[1] C. W. N. McGowan, R. C. Harvey, and J. W. Lowdon, "Oil Loading and Cargo Handling Facilities at Mina al-Ahmadi, Persian Gulf." Proc. Instn Civ. Engrs, Part II, vol. 1, p. 249 (June 1952).

[2] *Ibid.*, p. 309.

the gravity-type fender, in all the circumstances, was not preferable to the fenders incorporating rubber, faggots, or springs, all of which deteriorated with the passage of time.

Mr R. C. Harvey asked if the Author could give some of the mechanical properties of the rubber which was specified, and if he could state whether the ratio of the length to diameter for the cylinders or adhesive disks had been fixed arbitrarily or by analysis. It appeared that the Author had endeavoured to work to high shear stresses, but that if the cylinders had been longer he would have obtained the desired flexibility with less stress.

Mr Harvey had noticed that, in the two designs for the horizontal type of gravity fender, use was made of concrete with a specific gravity of about 3·3. It was not, in his opinion, reasonable—indeed, it was disrespectful to wave action—to place a lump of concrete of such a shape and of comparatively low specific gravity, suspended by means of hangers or bolts with nuts, in the open sea or in harbour conditions where a 4-foot wave was expected. It was his view, based largely on the working of the Kuwait piers, that the vertical fender was much better in the exposed positions which sometimes had to be used for the large commercial type of jetty which was being designed and constructed nowadays. The waves had little effect on such a steel or concrete cylinder because quite an appreciable part of the cylinder was out of the water, even at high tide, and the shape was more suitable. In any case, with a steel shell it was possible to use concrete suitably loaded with steel drillings, punchings, etc., so that a specific gravity of about 3 or 3·5 was obtained, which would make the fender almost immune from the effects of any waves. On that account, that type of gravity fender was probably preferable, except in sheltered water, to the horizontal type. Furthermore, all large ports nowadays were equipped, as part of their ancillary craft, with floating cranes which had heavy lifting capacities, and the lifting of a vertical cylinder weighing 40 to 50 tons did not present much difficulty to a port so equipped. In fact, with one spare cylinder available quite a big breakdown could be put right in a very short time. The time a berth was not available for use was a very important factor in commercial port working.

*** **Mr P. R. Robinson** referred to the Author's statement that damage to fenders was almost always caused by forces acting parallel to the berth rather than normal to it—except in the case of an outright accident. That was, of course, borne out by practical experience. Nevertheless, many fender designs seemed to suggest that the forces parallel to the berth were regarded rather like somewhat disreputable relations, whose existence unfortunately could not be ignored but who were not really nice to know. It might therefore be interesting to consider what the relative proportions might be of forces acting parallel to and normal to the berthing face. To do that, some assumptions on speed and angle of approach were necessary,

*** This contribution was submitted in writing.—SEC. I.C.E.

and fairly common ones—in the case of tankers at least—were that the worst conditions in which a ship under control would make initial contact with a berth obtained when the ship approached at an angle of $12\frac{1}{2}$–15 degrees between the axis of the ship and the berthing face, and at a speed of $\frac{3}{4}$ to 1 knot along that axis. Such a berthing would not of course be a good one, but it was of the nature which might be expected to occur perhaps once in fifty berthings, or at any rate sufficiently frequently to require to be catered for in the design—though the exposure of the berth and other factors had a large bearing on the frequency. Taking the Author's 40,000-ton ship, it would be found that on the foregoing basis the speed of approach normal to the berth was nearly $\frac{1}{2}$ foot per second and the total kinetic energy in that direction was about 1,500 inch-tons, which, on the Author's assumption of half the weight being operative at the point of contact, gave about 750 inch-tons to be taken by the fenders at that point; that was of the same order as the 1,000 inch-tons noted by the Author. The speed of the ship parallel to the berth, however, was about 1·6 foot per second, and its total kinetic energy in that direction was no less than 19,000 inch-tons. Mr Robinson did not, of course, suggest that that energy needed to be absorbed by fendering, because the ship was eventually brought to rest in that direction by other means, nor that all that energy could be transmitted to the fendering, except perhaps in the case of an outright accident, and still less that it would be economically justifiable to design the fendering to absorb it. He did feel, however, that that figure of 19,000 inch-tons served to underline the magnitude of the forces which could exist parallel to the berth, and which were the forces usually causing damage to the fendering. The amount of that energy which could be transmitted to the fendering was, he felt, a matter of conjecture, because it depended on whether it was transmitted by friction or by the engagement of a projection on the ship's side with the fender, or both. It would, however, be obvious that, whatever the proportion of the energy that was so transmitted, its effect would be not merely to tend to cause a lateral movement of the fender, but also to introduce into it a twist of some sort. The use of rubbing strips which could be fairly easily torn off was not entirely a satisfactory answer, and it would seem more logical to design the fender so that the point of initial contact between the ship and the fender face could rotate about a vertical axis, or could otherwise move, so that any engagement of projections on the ship's side with the fender could be released before the fender was damaged. In any case it would be apparent that it was as necessary to take into account, when designing fenders, the potential forces parallel to the berthing face as it was to provide for the absorption of energy normal to it.

The financial loss caused by a berth being out of use through damage to fendering varied greatly, but—again taking tanker berths as an instance, though perhaps an extreme one—where berths were used intensively by tankers, as they very often were, there might be two or even more berthings

per week, or say between 100 and 150 per year per berth. If there was one berthing in 50 under the conditions mentioned above and if the fendering was not designed to cater for such conditions, it was liable to be damaged two or three times a year. If each occasion put the berth out of action for only 1 day the demurrage charges on the tankers alone (at £1,000 per day for the large ships) would be £2,000 or £3,000 per year, without considering any concomitant loss to the oil-operating concern, which might be much greater.

Finally, Mr Robinson felt that, in general, civil engineers did not really have accurate information on the conditions for which they had to provide in the design of fendering. For example, there was little information on the speeds and angles of approach which did in fact occur, nor on the extent to which modern ship design would permit deformation of the hull to absorb some of the energy on impact. The present somewhat hit-or-miss basis of fender design was becoming increasingly unsatisfactory with the growing economic importance of the efficient use of berths, and he suggested that the matter was one which might usefully be considered jointly by the Institution, naval architects, marine superintendents, and other representatives of shipping concerns and port authorities.

The Author, in reply, thanked the various speakers and said that considering the rude things which he had said on other occasions he had been treated very kindly. His remarks on Maritime Paper No. 19 when he referred to the " funeral service " had sounded a little clever at the time but they did not sound so clever now. Professor Baker had been a particular friend of his then, and still was, but the Author did not propose to withdraw the remark until he had seen evidence of jetties built after the one at Kuwait. When someone started to use them again in a big way, then he promised to withdraw the remark.

Various speeches had covered similar questions and he proposed to reply under subject headings rather than to individuals.

Rubber.—The life of rubber itself seemed satisfactorily proved since rubber blocks used on jetties at Colombo and Singapore were still in good condition after 16 years. Experience of bonding for fenders was limited to 2 years and whether or not the risk of damage to the bond by rusting of steel plates was serious had yet to be established. Design No. 15 (*Figs 18*) had not been built. Very little full-scale testing of large blocks of rubber in shear had yet been done. Such tests were being put in hand and it was hoped to have results soon. In the absence of absolute tests—especially to shear angles of 45 degrees—specialists had advised the maximum shear stress of 160 lb. per square inch as quoted in the Appendix and the dimensions of units had been based on that. It was probable that the kinetic-energy capacities were on the low side but even if energy in compression was twice as great as in shear the question of use in jetties would have to be considered. Design No. 1 (*Figs 1*) had no longitudinal resilience ; that in *Fig. 37* used three blocks of rubber and therefore 50 per cent more than an

equivalent shear design; and all the rubber in *Figs 36* was not effective for blows in all directions. If Mr Cuerel had been able to devise a means of using rubber in compression which overcame those drawbacks, it would be most interesting if he would publish it. The Author believed, however, that the jetty to which he referred had a low-water bracing deck and if that was necessitated by the fender design the extra cost of that tidal work should be debited against the fender. Deflexion with rubber might not exceed 12 inches and the complementary high thrust might also add to the cost of the main jetty structure.

Gravity fenders.—The Author's main objection to vertical gravity fenders was that they required supporting at or below low water and made the jetty structure more costly than it needed to be. No damage of any kind had yet been sustained by designs Nos 9 and 10 but it had been reported that each on occasions had been pushed back about 75 per cent of their travel. It could not be emphasized too strongly, however, that those designs had no longitudinal resilience in themselves and were suited only to the special berthing conditions which obtained in Admiralty work, that was to say, where very substantial floating fenders were always used. Longitudinal resilience could probably best be provided by using slings which raked on elevation. It was interesting to learn that that had already been done and that no troubles due to sudden snatch in the slings had been experienced. In spite of that the Author considered that some form of spring should be incorporated into such raking slings in order to counteract that tendency. The individual weight of the largest gravity fender used by the Author to date was 25 tons. Weights of 45 tons were about to be built but even those did not necessitate extra piles to carry them and the estimated cost of providing 1,000 inch-tons in that way was £2,500. A suspended gravity fender of 150 tons had never been contemplated by the Author.

Timber.—Table 1 indicated that the kinetic-energy capacity of timber was high. Greenheart was stiffer than elm or Douglas fir but was much stronger, and the ultimate kinetic-energy capacity and deflexion of a greenheart dolphin, where long supported lengths were possible, were quite remarkable. Normally the most important characteristic of greenheart was its resistance to attack, but where high kinetic-energy values were required its great ultimate strength was important. Admiralty berthing authorities always required vertical rubbing timbers as standard practice.

Other designs.—*Figs 25* appeared attractive at first glance but resilience came from the piles only. The heavy reinforced-concrete beam did little work and served mainly to bring more piles into action. In the sketch, piles were shown at an average spacing of 4 feet and from Table 1, even with pile lengths of 60 feet, an ultimate kinetic-energy value (within the elastic range) of 50 inch-tons per pile was high. Allowing for some reduction due to direct load, the maximum average capacity was hardly more than 10 inch-tons per linear foot. To achieve that, each pile would have to

deflect 24 inches and it was difficult to imagine the heavy reinforced-concrete capping beam "snaking" to that extent along its length. In use it would seem that such a design would be far too rigid for light craft and it would be interesting to learn how it did in fact behave. The cost, as sketched, would probably be £100 per linear foot.

Fig. 26, like other similar proposals which had appeared in the technical press recently, seemed somewhat elaborate for maritime work and the Author felt it would need to be substantiated by comprehensive full-scale trials.

Spacing of fenders.—The Author fully agreed that continuous protection should be provided where possible. Cost considerations dictated that that could be only a relatively low figure per linear foot and the problem of distribution arose, as in *Figs 25*, when large individual blows had to be designed for. With jetties taking large vessels as an almost continuous function, the provision of isolated fenders of high individual kinetic-energy capacity seemed to be satisfactory. In effect, the Admiralty practice of using heavy floating fenders for nearly all berthings meant that even solid wharf-walls had isolated fender points along their length. With very mixed traffic such fenders would be exposed to the risk of small vessels getting into the side of them but it was doubtful if the spacing would have much effect on that.

Blows on vessels.—The vulnerability of vessels to blows was seldom a limiting factor in the Admiralty, and the Author had little experience of that. An individual load of 100 tons on the jetty structure was a reasonable upper limit to aim at and if, as with gravity fenders, a movement of 40 inches could be provided, the load would not have to exceed about 80 tons for a kinetic-energy of 1,000 inch-tons. Somewhat limited experience with large oil tankers suggested that that was quite acceptable to those relatively vulnerable craft, even when applied through a floating fender to design No. 10 (*Figs 12*).

Costs generally.—The Author agreed that costs of fendering were becoming an appreciable proportion of total jetty costs and that that point should not be overlooked. Possibly there had been a tendency since the war to provide more resilience than was necessary, but as a rule good fendering should reduce the cost of the jetty superstructure, and the overall economies of usage of the completed jetty had also to be considered.

Speed of approach.—There was no authoritative data on speed of approach and probably never would be. When really resilient fendering was provided there was the definite likelihood that masters would deliberately increase their speed of approach when they realized that the fendering allowed it. Because of that the Author felt that arguments on angles of approach and effective weight of vessels were perhaps unjustified refinements. Particular cases could arise when exact requirements could be stated—the Author knew of one in which the full weight of a 4,000-ton vessel was expected to be effective at 1½ knot—but in general half the

weight of the vessel at $\frac{1}{2}$ foot per second seemed to give a reasonable design basis.

The Author believed that durations of loads had to be very low indeed before their effects were markedly different from those of static loads. A duration of $\frac{1}{16}$ second in that respect was actually very high.

Correspondence on the foregoing Paper is closed and no further contributions, other than those already received at the Institution, can now be accepted.—SEC. I.C.E.

RAILWAY ENGINEERING DIVISION MEETING

11 November, 1952

Mr J. C. L. Train, Member, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Railway Paper No. 47

"Some Major Problems in Railway Civil Engineering Maintenance"

by

Alfred Henry Cantrell, B.Sc.(Eng.), M.I.C.E.

SYNOPSIS

One of the most important functions of railway civil engineering is the efficient maintenance of the permanent way and structures. There are, however, numerous problems, many of which disappear as a result of good organization, but many technical difficulties are present which vary to some extent in nearly every case.

The Author draws attention to the fact that as a result of loads gradually increasing on railways, and many structures nearing the end of their useful life, many maintenance problems are having to be solved. Examples of these are given concerning permanent way, earthworks, bridges, tunnels, major roofs, etc., and solutions which have been adopted are described.

The greatest problem of all is the carrying out of these works with the minimum interference to the traffic, and this is probably the most important factor involved in deciding on the action to be taken.

Introduction

Civil engineering maintenance does not sound a very interesting subject to those who have no experience of it, especially to students finishing their course of theoretical training. The fact is, however, that the circumstances in which the work has to be carried out and the limited time which is usually available for doing it bring a great deal of interest to the problems furthermore, the engineer responsible for maintenance has to make decisions based on a combination of theoretical training, practical experience, and common sense.

This is probably more so in railway work than in other branches of civil engineering because interference with the smooth running of traffic must be as small as possible, but at the same time the weights and speeds of trains necessitate careful consideration being given to the strength of work which is put in. Because of vibrations set up by traffic special care

has to be taken in designing temporary works, such as timbering of excavations.

In most cases trains cannot be diverted and, whenever possible, work must be carried out without stopping them except possibly for a few hours in the middle of the night. The imposition of temporary speed restrictions can be allowed to a certain extent. Mechanical plant can be used only when there is no possibility of fouling passing traffic, or when precautions can be taken to ensure that it is clear before traffic approaches.

As a result of these difficulties, the cost of repairs is bound to be high and maintenance considerations become important factors in the design of new works. To the uninitiated, railway works often appear to be heavier than necessary, but in the long run this is more economical. Retaining walls supporting embankments carrying fast-moving traffic must be sufficiently strong to be able to withstand a high degree of compaction behind them, combined with vibrations in the material they are supporting. Buildings must be sound and robust so that they do not suffer from the shaking effects of passing trains. The steelwork of station roofs, sheds, and bridges over the line is exposed to the corrosive action of sulphur in engine smoke, and it is worth while allowing a little for this in the design, especially in thinner sections.

Maintenance problems are of infinite variation and it is impossible in this Paper to deal with more than a few of them. The Author therefore gives details of works on the Southern Region of British Railways with which he is familiar, but which are no doubt similar to those encountered elsewhere, with the object of bringing out the main principles involved.

They are considered under the following general headings :—

1. Permanent way.
2. Formation and foundations.
3. Slips.
4. Scour.
5. Tunnels.
6. Bridges and roofs.

Permanent Way

The problem of track maintenance is basically one of organization.

The maintenance of good line and level is the direct responsibility of the Length Ganger, who receives close supervision from his Permanent Way Inspector. These men are essentially practical and no amount of theory can replace the experience they gain over a period of many years. Nowadays, however, every endeavour is made to give them some theoretical knowledge by persuading them to attend special evening classes, and there is no doubt that as a result they take greater interest in their work and therefore produce better results, but the fact cannot be overlooked that

some of the best gangers are the older ones who have been brought up on the track since they first started work.

Technical supervision must be given by the District Engineer and his assistants. In the Author's opinion, by far the best way of doing this is for the District Engineer regularly to walk all the routes under his control, accompanied by the particular Inspector concerned. He is then able to see for himself that the standard he desires is being maintained, or where this is not so, he can take the necessary steps to ensure that an improvement is effected. Contact between the engineer and his men is invaluable, and many of the gang's difficulties—which would not otherwise be known—can be solved by discussion on the site. Inspection by trolley or by special coach cannot take the place of these walks, but certainly give a useful additional means of keeping in touch with conditions of way and works over a longer length of line in a shorter time.

The annual marking of lengths, now standard on British Railways, is a valuable additional means of helping the District Engineer to know the condition of the track for which he is responsible.

The theoretical aspect of track work deals more with junctions and connexions than plain line, but practical experience is essential in order to produce a lay-out which can be put in position in the time available and to give as few maintenance difficulties as possible afterwards.

Electrification and the increasing weight of modern traffic give a tremendous pounding to point-and-crossing work so that at very busy junctions too high a proportion of the gang's time is spent in keeping the various bolts and fastenings tight. At the same time the crossings themselves wear down quickly and, until a few years ago, replacements had to be put in frequently when traffic could be stopped for a time.

More recently, this latter trouble has been met by making good the wear with metal deposited on the worn surface by either the electric arc or oxy-acetylene welding process. Both the above types of trouble, however, are greatly reduced by making those parts of the junction where the greatest impact is experienced in cast high-manganese-steel material. As can be seen in *Figs 1* (facing p. 120), much fewer parts are required in the make-up of such crossings, and the ganger's work of keeping the bolts tight is consequently reduced considerably ; but in addition the better wearing qualities reduce the work involved in making up the worn surfaces by welding and the replacement of crossings when worn out.

A complete lay-out of this type was installed in 1944 at Borough Market Junction where traffic from Cannon Street and Charing Cross Stations converge. The original material is still in use, but previously crossings had to be changed or pulled back every few months, and much of the ganger's time was spent daily in tightening up the fastenings. Creep has been eliminated.

Other similar schemes were put in at Lewisham Junction in 1950 and Metropolitan Junction in 1951 and have given excellent results.

These lay-outs are, however, very expensive and can be justified only in places carrying very frequent and heavy traffic.

On British Railways, about 2,000 miles of track are renewed each year. Because of the shortage of labour which has been experienced since the war, methods have had to be adopted for doing much of this important work by machine, and a large mileage is now carried out by the pre-assembly method, using either cranes or track re-laying units. The latter are particularly useful in tunnels and were described by the Author in a previous Paper.[1]

To meet modern conditions, a great advance has been made in the use of various types of portable mechanical plant, but this is too wide a subject to be included in this Paper.

Formation and Foundations

A problem which is occurring to an increasing extent is the gradual softening of the clay under the track formation in certain areas. The cause differs from place to place, but generally it can be considered to be an insufficient depth of suitable load-spreading material between the underside of the sleepers and the clay subsoil, combined with an inadequate drainage system and the increase in weight of modern traffic. The effect is that the surface of the clay breaks down and turns into slurry which works its way up through the ballast and causes what are known as "pumping sleepers," or flows plastically outwards and upwards to form "heaves" at the sides of the track. The sleepers move up and down under traffic, and in wet weather it is impossible to pack them so that they remain firm for more than a few hours.

Once the foundation has started to soften, conditions gradually become worse, and a major operation is required to rectify matters. Various methods have been tried to achieve stability, such as :—

(*a*) By driving a series of pile "spuds" to form holes which are then filled with sand to act as a drainage system.

(*b*) By a method of "blanketing" between the clay and the ballast.

(*c*) By pressure-grouting under the formation.

The application of these methods depends on the particular site under consideration, but the Author is of the opinion that the most generally satisfactory one is blanketing. This involves taking up the track, removing the ballast and all other material including the damaged clay, down to a depth decided by a soil mechanics investigation to give a suitable spread of the dead and live loads, such that the new foundation will not fail. A usual but not invariable depth is 3 feet 6 inches below the top-of-sleeper

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[1] A. H. Cantrell, "Renewal of Ballast and Track by Mechanical Means in Polhill Tunnel." J. Instn Civ. Engrs, vol 31, p. 331 (Feb. 1949).

level. The excavation thus formed is filled by a "blanket" of a fine quarry dust followed by a depth of 3 or 4 inches of about ½-inch stone chippings and then standard track ballast. A drainage system alongside the track is also constructed. The dust prevents abrasion of the clay surface, and the chippings ensure that the largest ballast does not work into the dust.

Details of particular works of this type have been described in a previous Paper [1] presented to the Institution and are therefore not given by the Author.

SLIPS

Slips or minor landslides are a source of great trouble to the maintenance engineer. The problem is not necessarily caused by the amount of material involved, but usually by the difficulty of access to the site other than by rail and by the necessity for working with the least interference to railway traffic.

The classic example of a major landslide in Great Britain was at Folkestone Warren. *Fig. 2* (between pp. 120 and 121) gives a general view, and shows on the right the new face of the cliff after various major slips which have taken place during a period of 200 years or more. The movement is not entirely one which follows the well-known slip-circle type, for it is combined with a sliding forward of the general mass on a surface between lower greensand and gault. Movement has continued over recent years and repair works to arrest this have been put in hand. They consist in the main of a heavy mass-concrete apron and retaining wall on the foreshore and drainage tunnels from the toe, running under the railway to the main mass of the detritus behind.

The features of this landslide have been described by A. H. Toms,[2] and it is anticipated that another Paper will be presented in a future session describing the repair works which are being carried out.

The more usual problem of the maintenance engineer occurs in banks or cuttings of a clay nature. Fortunately such slips usually give plenty of warning before any serious movement takes place. A slight bulge in the slope of the earthworks combined with longitudinal cracks are obvious indications to staff who are observant. Another excellent warning is a gradual local rise or fall of rail level.

An early decision must be made on future action. Once failure of the bank has started, slow but powerful movements take place. It is better to let the movement follow its natural course than take half-measures to

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[1] A. H. Toms and W. F. Beatty, "Remedial Measures for the Improvement of Railway Track Formations." Railway Paper No. 37, Instn Civ. Engrs, 1950.

[2] A. H. Toms, "Folkestone Warren Landslips: Researches carried out in 1939 by the Southern Railway Company." Railway Paper No. 19, Instn Civ. Engrs, 1946.

stop it, especially in a cutting. Repair works which fail may impede the work of clearing the line after the slip has moved on to the formation.

Ordinary slips can be divided into two classes :—

(1) Those in which the slip circle comes to the surface on the slope of the bank.
(2) Those in which the failure is more deep seated.

In both cases it is necessary to stop the movement at the toe of the slip or bank.

In the former type the easiest way when the slip circle emerges above but near the bottom of the slope is to build a thick block stone wall and place behind it sufficient heavy material to form either a fairly flat slope or a berm, thus putting a considerable weight on the toe of the slip. If the failure is higher up the slope, it may be better to let it take its course, because on complete failure it is not likely that a great quantity of material will have to be taken out of the cutting, or put in the embankment.

When the slip plane comes to the surface outside the toe of the slope, it is possible to stabilize the position by weighting the toe. This is difficult in cuttings, since no appreciable material can be placed without fouling the track, but at the bottom of banks, slips can often be prevented by taking action as soon as any sign of movement has been noticed. The best way when site conditions permit is to weight the toe, but care must be taken that in the process of doing this the materials are not placed, even temporarily, high up the slope, or the tendency to slip will be increased. The Author on one occasion managed to stop a slip which had started to give trouble by unloading some scrap concrete piles and paving slabs at the bottom of a bank so that the load on the toe was sufficient to stop the movement. In this case private gardens extended to within about 2 feet of the bottom of the embankment and no room was available for any other immediate loading without taking over a considerable part of the small gardens. On another occasion, suspecting that a certain troublesome embankment about 30 feet high would probably move during the next winter season, arrangements were made for dirty ballast from track renewal work in a tunnel to be unloaded down the side of the slope to form a heavy berm at the bottom. The ballast stood at a steeper angle of repose than the bank material and therefore it was possible to place a considerable load on the toe. Unfortunately, the wet season came earlier than expected, and the slip moved, but the preventive works were sufficiently far advanced to minimize the movement, and the track was kept open with the imposition of a speed restriction only. If the weighting had not been carried out, there is no doubt the track would have been blocked for several days and the slip would have moved on to a row of houses, which were only about 10 feet away. Permanent repair works consisting of a heavy concrete wall along the line of the original bottom of the bank are now being carried out.

The Author is of the opinion that the driving of piles for slip prevention must be adopted only after careful consideration. If it appears that the bank is about to start moving, the vibration set up by this method may be just sufficient to bring it down. Further, if the piles can be driven only where the slip plane is fairly deep, the cantilever action by which they stop further movement will require a very high resistance to compression in the top of the undisturbed material below the slip plane. In some cases this would be excessive, and even if no difficulty is experienced soon after driving, a gradual movement can take place over a period of years which, in due course, would give trouble.

When sheet-piling is driven, a sound job can be produced by leaving it projecting out of the ground a sufficient distance to form the front of a concrete retaining wall which will enable filling, and therefore more weight, to be placed on the lowest part of the slip.

In deciding on preventive measures or repairs, the fact must not be overlooked that the presence of water is a prime factor in causing slips. A satisfactory scheme must therefore include allowances for drainage or for preventing water reaching the weakened earthwork.

The most usual method is that of constructing drainage legs. These consist of trenches up the slope of the bank filled with hand-packed hard-core. Sometimes they are made to fork part of the way up the slope to give a Y formation in plan. An important feature, however, is that they should be built deep enough to reach the slip surface, so as to lead away any water which may be there. If the trench is in clay, it is advisable to put a thickness of granite dust or similar fine material between the hardcore and the side of the excavation. This prevents the clay from gradually squeezing into the cavities of the hardcore, thus ensuring that the drainage remains effective. The bottom of the legs must be connected to an efficient system of drains.

This type of construction requires an appreciable time to complete. More immediate results can be obtained by thrust boring at a slightly upward slope into the side of the slip, especially in those instances where it is suspected that a pocket of water exists. It may be necessary to drive two or three pipes before the right location is tapped, but when this has been done, an immediate easement in the movement of the slip takes place.

Where a cutting has been built along the side of a natural slope, trouble is sometimes caused by the natural flow of surface water towards the railway. The difficulty is often overcome by digging a ditch along the uphill side of the cutting, but it is advisable to line it with concrete. If this is not done in clay formations cracks will form in the bottom of the ditch in dry weather so that when the rain comes, water is led below ground in just the right position for starting a slip.

Scour

Scour does not give trouble to the railway maintenance staff in Great Britain so often as slips, but since it is difficult to see its encroachment, especially on underwater foundations, care in observing signs of it is important.

Prevention, as for slips, is much better than cure. For big river crossings it is advisable to have detailed soundings taken at regular intervals. At other places proddings can be very helpful, but the local gang must also keep a watchful eye on any tendency to change course that the stream may have, especially if there is the possibility of it working behind a wing wall or causing a slip in an embankment.

In such cases protection can be given by driving a line of sheet piling, by building a revetment of bags filled with concrete, or by placing stone pitching on river banks and round piers.

Another useful means of giving protection which does not seem to be adopted in Great Britain is that of using gabions. These consist of cages made of fairly heavy galvanized-wire mesh of convenient size, but as big as possible for the work in hand, which are subsequently filled with boulders or lumps of rock of varying sizes, such as can be handled by a man.

The cage which in the first place need consist only of a bottom and four sides, is first put in the position where it is required. It is then filled with boulders or rock, the best result being obtained if they are hand packed. A lid is next wired down to the four sides of the cage and the result is in effect one large mass of stone which has been placed in position without a crane.

During the 1939–45 war the Author successfully used this device in Italy. A three-span bridge over the River Ofanto was demolished by the retreating Germans. It was necessary to reconstruct it as quickly as possible. Since the pier foundations had not been damaged very seriously, it was decided to build on them in order to save a considerable time. Local information, however, was that a few weeks later the river, which was then not much more than a trickle, would rapidly rise many feet, and would probably wash away the damaged foundations.

Pile-driving equipment was not available and the bed of the stream was littered with heavy pieces of masonry from the demolished bridge so that any work on the foundations would have been slow. In order, therefore, to reduce the rate of flow of the water, a dam was built about 50 or 60 feet downstream. It was about 10 feet high and was constructed of gabions about 6 feet long by 3 feet square section. When the river rose, the dam stood, and the foundations of the bridge gave no trouble.

On another occasion, a temporary bridge was put out of action by floods rising 18 feet overnight and undercutting one of the abutments. The water level soon dropped again and after the damage was repaired,

the bank was lined from bottom to top with gabions. Next time the water rose there were no serious effects.

In situations where there is always an appreciable depth of water, gabions can be made at a higher level, such as on a bridge or riverside wall, pushed over, and dropped into position. Once down, they cannot be moved, however, so the operation must be done with care. Slightly greater control, if it is necessary, can be obtained by assembling the gabions on a barge or floating platform and dropping them into position.

A valuable application of this device is for providing protection from the sea to breakwater foundations and other similar structures alongside which it is not necessary for boats to tie up. The gabions can be built on top of the wall and pushed over, but in order to resist the powerful action of the waves, they need to be bigger than for protection works in rivers. A great increase in efficiency is obtained in this sort of work if the stones are put into the cages only loosely and the lids fastened down before they are quite full. When this type is dropped into position, they bed down better on an irregular surface, so that the sea cannot force its way under so easily.

Tunnels

The time has now been reached when many of the older tunnels on British Railways require a considerable amount of expenditure to keep them in good condition. Two recent Papers [1,2] described repair works which had become urgently necessary to prevent the almost certain collapse of two tunnels, one near Hastings, and the other near Nuneaton.

So often the cause of trouble is the relatively small bearing area allowed for the side walls, or the inadequate resistance provided against the pressure behind them.

It is not suggested that the engineers who built them could be blamed, for the Author has the greatest respect and admiration for the work of those pioneers who built tunnels more than 100 years ago. The greater present-day knowledge of this type of work has, to a large extent, been gained by studying the maintenance difficulties encountered since their time.

Much of the trouble in these tunnels can be rectified by underpinning the sidewalls or strutting between them or a combination of both methods.

As a result of the necessity to interfere as little as possible with the running of traffic, this type of work needs careful design and organization. That carried out in Bo-Peep Tunnel was in a location where serious movement had taken place and a very considerable strengthening of the structure had to be put in hand, and complete possession was required.

[1] F. E. Campion, "Part Reconstruction of Bo-Peep Tunnel at St Leonards-on-Sea." J. Instn Civ. Engrs, vol. 36, p. 52 (March 1951).

[2] C. W. King, "Arley Tunnel: Remedial Works following Subsidence." *Loc. cit.* p. 76.

In some cases the movement is not so great and the type of scheme shown in Figs 3, Plate 1, can be adopted. This was carried out in Warrior Square Tunnel, Hastings, where the walls had started to move inwards for a length of about 1 chain, but no vertical movement seemed to have taken place recently, although there were signs of such having occurred in the past. However, to ensure stability, it was decided that underpinning of the walls should be carried out while the opportunity existed.

By putting in crossover roads it was possible to maintain single-line traffic past the site, first over one track and at a later stage over the other, except that at the beginning of the work a week-end possession of the lines was required to cast three of the concrete struts across the tunnel from wall to wall to prevent any inward movement taking place when the later stage of excavation would reduce the lateral support given to the inside of the walls.

The work was divided into a series of short lengths by the struts, which were 8 feet apart. With one track removed, two or three sections were taken in hand at the same time, for half the width of the tunnel, but they were separated by at least four undisturbed or completed bays. Traffic was then changed over while the other half width was constructed. The temporary crossover roads were put in during December 1951, and normal traffic restored in March 1952.

A different type of trouble is encountered when the lining of a tunnel starts to bulge, which often occurs about 10 feet or more above rail level. It usually necessitates the cutting out and rebuilding of the affected brick-work, possibly extending up to the crown. This in itself would be no difficult task were it not for the necessity of maintaining traffic through the tunnel.

In these cases timber lagging is required, but the method of support depends upon the clearance available. In a double-line tunnel, especially if it is of any great length, the two tracks must be kept open for traffic if at all possible. The design for the supporting ribs should therefore receive very careful consideration so that sufficient clearance is available for passing trains, the most difficult point being at the cornice of coaches and engine cabs.

A design which was used recently on the Southern Region is shown in Figs 4, Plate 1. When using steel centering it is preferable that it should be adjustable to a certain extent so that when it is taken down from one position it can easily be erected in its next position, where the tunnel profile will most probably be slightly different.

If the clearance is not quite sufficient, it is sometimes possible to obtain it by slewing the track or tracks towards the 6-foot way, but the standard minimum must be carefully preserved.

When sufficient clearance cannot be obtained, single-line working must be introduced. The engineer's work is thus simplified and timbering as shown in *Fig. 5* (between pp. 120 and 121) can be employed. This was used in

Higham Tunnel before the war and is of interest because of the unusual shape of the barrel. It is very high, wide at rail level, but narrow at cornice level. The standard 6 feet 5½ inches between the running edges of the tracks is reduced to 5 feet 11½ inches, which is only 2 inches more than the minimum allowed. The tunnel is 1,507 yards long and the repairs were required about half-way through it. It is between signal boxes which are 2½ miles apart, and another tunnel, 1 mile 585 yards long, is also situated between these limits. Single-line working of the normal type with temporary connexions sufficiently near the boxes for easy operation would be so long that it would cause chaos to the heavy traffic passing. The tracks were therefore interlaced over a short distance covering the work, thus avoiding the necessity of having turn-outs, so that with the help of track-circuiting it was possible to maintain the normal traffic with only little interference.

It is of interest to note that this tunnel and the adjacent one were built, before railways existed, to carry a canal between Gravesend on the Thames and Strood on the Medway. It will be realized therefore that although their shape is unsuitable for railways, it was quite satisfactory for the purpose for which they were built.

Another problem which will face maintenance engineers to a greater extent as time goes on is the making good of the inside course of brickwork in some tunnels, where scaling and deterioration of mortar has been intensified by the combined action of the sulphurous smoke from engines and water percolating through from the surrounding strata. The best scheme may be to cut out the remains of the disintegrated course and rebuild with good bricks, but it will be a long and tedious process and will require falsework, which can, however, be of fairly light construction. Resurfacing by the process known as "guniting" is another method, but in this case it is essential to clean off the old work and make sure only sound bricks remain. When this has been done, it may be found that the thickness to be made up is not suitable for the adoption of this method. The Author stresses the necessity of ensuring the good quality of the brickwork under the guniting. The process was used in Oxted tunnel in 1920, but it is now falling away in places and pulling with it what could be described as thick scales of brickwork. This is attributable to the fact that in the construction of the tunnel, bricks made on the site were used, but they were not of a long-lasting quality.

It is, of course, possible that a good repair could be made by incorporating mesh reinforcement with the gunite, but the restricted facilities for work may create difficulties.

Sometimes the portals of tunnels move bodily, pulling a short length of the lining with them. This may be caused by poor foundations or by the parapet and wings being too weak to act as retaining walls. In either case, very big forces must be involved, as is shown by the complete fracture of the tunnel brickwork.

It is of little use building buttresses to stop this movement unless they

are very substantial. They must have good foundations, be heavy, and have a reasonably flat angle, otherwise they will lift or heel over.

Bridges and Roofs

As in the case of tunnels, the problem of the maintenance of bridges is becoming more and more difficult as time passes, chiefly as a result of the structures becoming older and the increasing weight of modern traffic. The classic timber bridges constructed by I. K. Brunel, although excellent for their purpose for many years, have had to be replaced. Old metal bridges are gradually being reconstructed, and many underbridges have been rebuilt in order to meet the requirements of road traffic, to give either greater headroom or to allow for a wider roadway. Masonry bridges are giving much better service and in most cases are showing no signs of decay or overstress.

There are many cases of bridges which have had to be replaced because of corrosion of the metal, and in industrial areas this is becoming a serious problem, especially at the present time of acute shortage of steel. Many were constructed during the chief railway-building era of the nineteenth century, and they are now becoming age-expired at about the same time. It is, of course, possible to repair the corroded parts of many of them in situ, but this is of necessity a slow process if interference with traffic is to be reduced to the minimum.

The usual types of repair can be classified generally as follows :—

(*a*) Main girder repairs.
(*b*) Floor repairs.
(*c*) Viaduct repairs.

The subject is a very wide one, but an indication of some of the methods which have been adopted, without being by any means complete, is given below. Ways of carrying out the work have changed considerably for steelwork repairs as a result of the extended use of welding. In many cases this has eliminated the more laborious method of detailed measurement and templeting which was required when new plates had to be added to register with old rivet spacing, often involving the knocking-out and replacement of a large number of rivets. Additional plates can now be placed by welding, but it is essential to ascertain that the metal to which they are being fixed is sound and strong enough to transmit the stresses and also a careful welding technique must be employed to prevent distortion and to control the effects of locked-up stresses.

Main Girder Repairs

Main girder repairs are usually required in order to make up for corrosion of the edges of flange plates, or of the end web-plates and stiffeners of plate girders, especially of spans over heavily used steam lines.

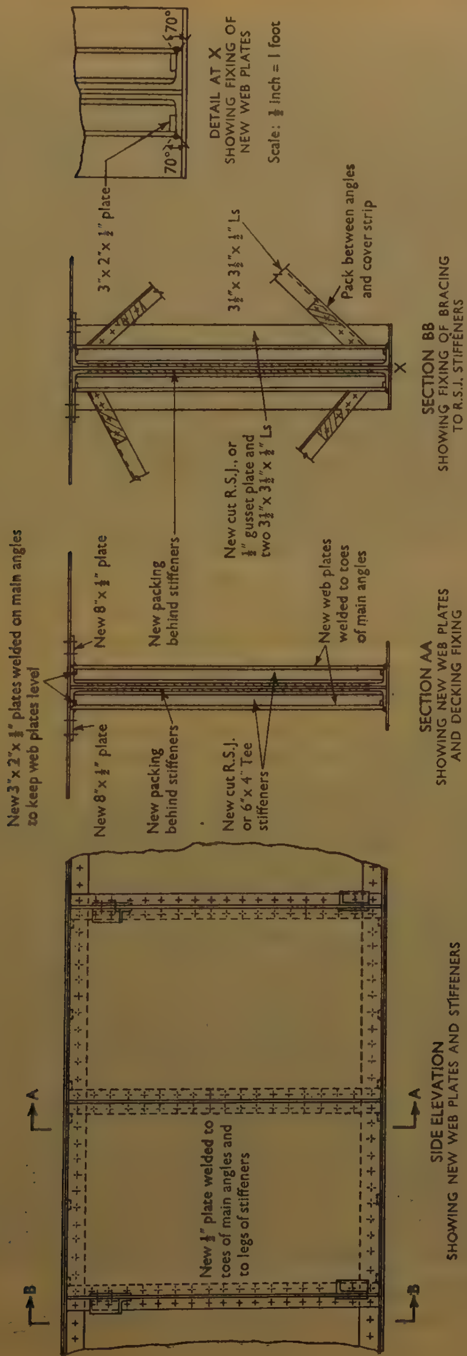
An example of this type of work is shown in *Figs 6*, which give details of repairs to a bridge carrying the Victoria lines over Battersea Goods Yard. Engines constantly shunt and often stand under the bridge, and, owing to its deck type of construction, smoke fumes are trapped and remain between the girders for many hours. Careful consideration was given to the complete rebuilding of the bridge, which carries four running lines and three sidings over three spans of about 85 feet on the skew, but owing to the intense traffic over the bridge, which would make reconstruction very difficult, it was decided to put in hand repairs which could be carried out with less interference, but unfortunately would take much longer. The web plates had suffered badly, and in some places it was almost possible to climb through the holes. The bottom flanges, thanks to excellent bridge composition placed there many years ago, were in good condition and therefore needed little attention, but much of the remainder of the girders were more or less rebuilt by welding on new plates. A new deck was built in some spans, incorporating ventilating grills.

On some parts of the original lines of the former Southern Railway, many wrought-iron plate-girder bridges were constructed with their ends built in. A time came when some of these showed signs of distress, in that severe movement was observed on the bearings and the surrounding brickwork cracked badly. When the ends were uncovered, it was found that the girders had corroded to such an extent that some end stiffeners had almost disappeared, and some web plates were badly holed. The girder ends were therefore rebuilt in situ, a riveted construction being adopted since the welding technique for wrought iron had not been perfected at that time.

Some other similar bridges were then uncovered and examined, but the conditions found were generally not so bad. In view of the large numbers of spans of the same type which existed, some of them carrying a dozen or more tracks, it was decided not to start a general scheme for dealing with all such girders, but to observe them specially, because it was considered that any signs of failure would be seen long before action became urgent.

Flange corrosion is usually more easily repaired, because it is more accessible. A common method employed is that of cutting away the thin corroded edges of the outside flange plates, and welding up a new plate, holed to miss the rivets. The main welds are of the fillet type between the cut edges of the old plate and the surface of the new one, which must therefore be wider than that to which it is being fastened. A small fillet weld should be deposited round the insides of the holes cut to miss the rivets, in order to help in stress distribution and to prevent corrosion starting between the two plates by moisture and fumes getting in.

Figs 6



Scale: ½ Inch = 1 foot

IN-SITU REBUILDING OF GIRDER WEBS

Floor Repairs

When consideration is being given to the carrying of heavier traffic, it is sometimes found that the main girders of a bridge are sufficiently strong, but the floor is inadequate. A strengthening in situ in such cases is sometimes out of the question because headroom for staging or space for trestling, which would be required for long periods, is not available. A reconstruction of the floor has then to be undertaken.

A large scheme of this type was carried out on one of the widenings of the Charing Cross Bridge over the River Thames. This is shown in *Figs 7, 8,* and *9.* The original cross-girders were retained, but the timber rail bearers and decking were replaced in steel. *Fig. 7*, where a portion of the floor has been removed, shows the depth of timber in the old rail-bearers and *Fig. 8* shows the end of a section of the new floor. In *Fig. 9*, the new and old arrangements can be seen from above. The rail level and alignment had previously been far from satisfactory, and the opportunity was taken to improve both. Two electric gantry cranes able to travel from one end of the work to the other were erected on top of the main girders. By their use, work was made comparatively easy, for material trains at either end of a section of floor could be unloaded or loaded up without delay, in spite of the break in the continuity of the tracks. The ordinary type of railway crane could not have been used because the high main girders immediately adjacent to the track would have prevented it from slewing. The lines were closed only at week-ends, when the three tracks carried by the remainder of the bridge were sufficient for traffic purposes. Several complete bays of old floor, each 11 feet long, were taken out and the new steelwork put in each week-end, preparatory work, riveting, and a certain amount of waterproofing being carried out between trains or under shorter occupation of the tracks during the week. At first, four bays were done in one week-end but this was later increased to six. The work was started in July 1948, and finished in August 1949. The total length of decking was 1,009 feet.

Another interesting floor repair is being carried out on the bridge which carries the Southern Region lines over the Thames at Staines. This structure is of three through-type spans each 93 feet long, consisting of three plate girders carrying two tracks. Severe corrosion had taken place at the cross-girder connexions, such that in some cases their bottom flange angles had been reduced to paper thickness.

Repairs were carried out by welding additional plates or packings in place of parts which were cut out. Great care had to be exercised in the design of the details, because the old structure was made of wrought iron. Except for sealing welds, all welds on the original metal are so placed that they pick up the fibres of the wrought iron as shown in *Figs 10.* A fillet of stress-carrying weld-metal on the surface of wrought iron often pulls off the top laminations.

The wrought iron was chamfered to an angle of 45 degrees, and a

Fig. 1 (a)

OLD TYPE OF BUILT-UP JUNCTION

Fig. 1 (b)

REPLACEMENT INCORPORATING CAST-MANGANESE STEEL. VIEW TAKEN IN LAY-OUT YARD

Fig. 2

FOLKESTONE WARREN LANDSLIDE AFTER REBUILDING OF THE RAILWAY

Fig. 5

TIMBERING IN HIGHAM TUNNEL

Fig. 7

CHARING CROSS BRIDGE, SHOWING OLD TYPE OF FLOOR

Fig. 8

CHARING CROSS BRIDGE, SHOWING NEW FLOOR

Fig. 9

Charing Cross Bridge
New Floor in Foreground

Fig. 13

Fig. 14

Charing Cross Bridge

Bands Round a Cylinder

Joint in Bands

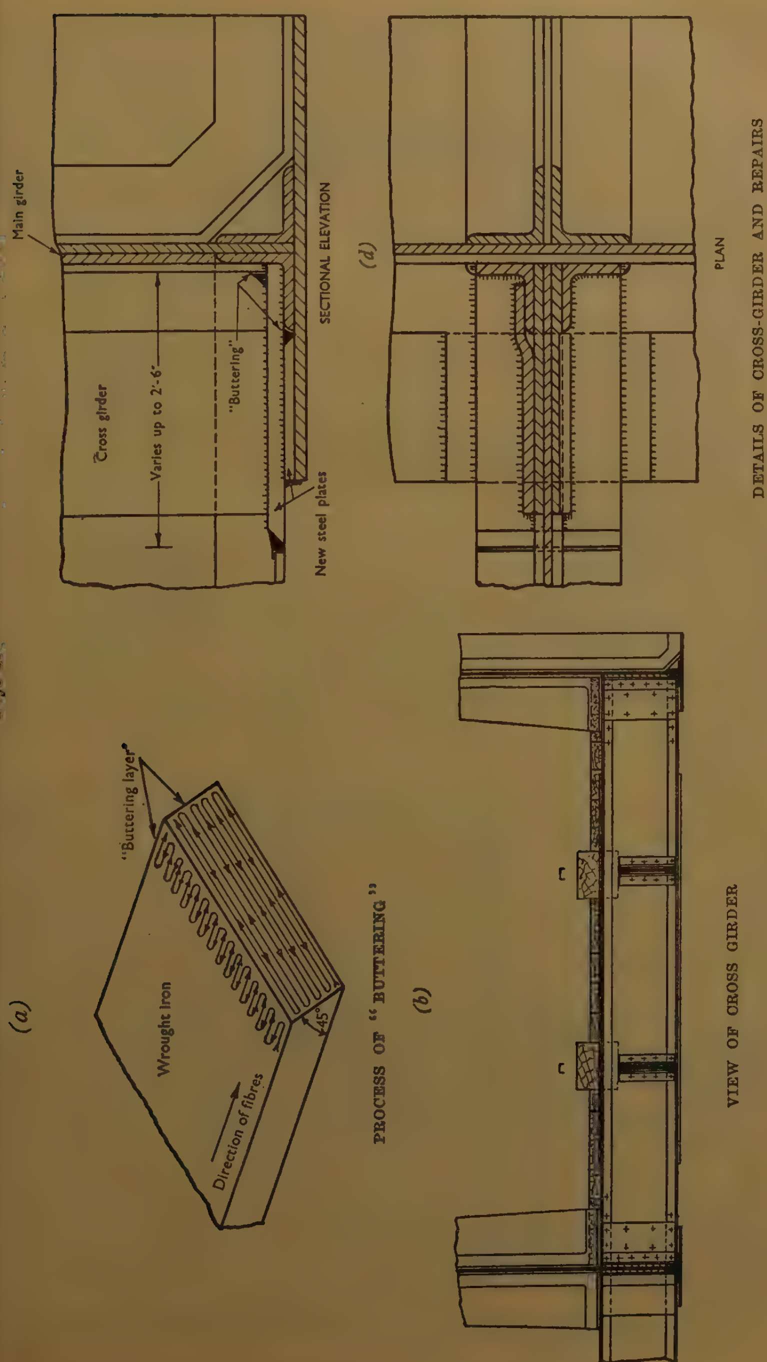


Fig. 10

(a) PROCESS OF "BUTTERING"

(b) VIEW OF CROSS GIRDER

(c) SECTIONAL ELEVATION

(d) PLAN

DETAILS OF CROSS-GIRDER AND REPAIRS

CROSS-GIRDER REPAIRS TO BRIDGE OVER RIVER THAMES AT STAINES

"buttering" layer of metal was deposited transverse to the fibres of the iron from a 10-gauge electrode with a current of 90 amperes, the temperature of the work not being allowed to exceed 150° C. When this "buttering" had been completed, the welding to the new steel was carried out with an 8-gauge electrode at a current of 140 amperes, the temperature control remaining as before. In normal work this corresponds to a consumption of fifteen electrodes per welder-hour. *Fig. 10* (*a*) indicates the method of applying the "buttering" weld metal.

Viaducts

Generally, brick or stone viaducts and arches have given little trouble on railways except in colliery areas, where subsidences present a very difficult problem. Many viaducts built in the earliest days of railways are now carrying heavy modern traffic without trouble, but some have shown signs of over-stressing. In many cases the cause has been found to be due to bad construction, not design.

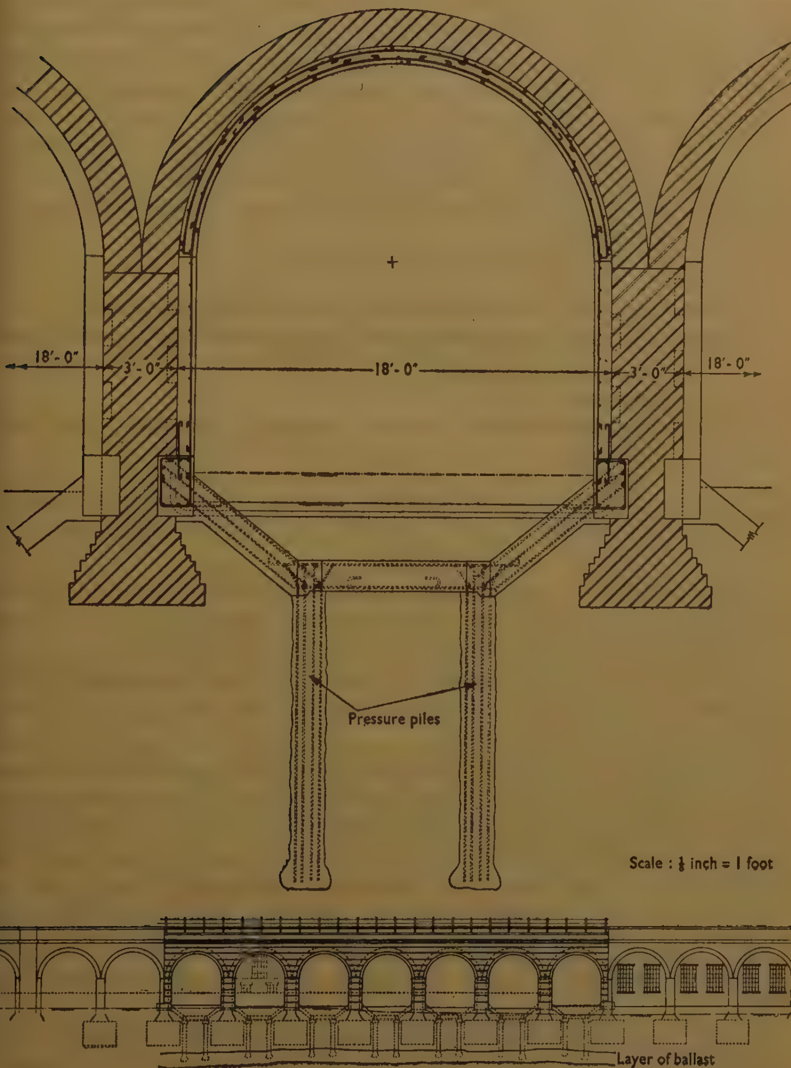
The original viaduct of the London-to-Greenwich railway, built in 1835, is satisfactorily carrying the Southern Region electric and steam traffic out of London Bridge Station, and no difficulties are experienced, but one of the several widenings built many years later has shown signs of failure. Upon investigation it was found that although the surface brickwork of the piers was sound, the interior was loose and hardly any mortar had been used. Repairs were carried out by erecting heavy timber centering under the arches either side of a pier, which was then rebuilt in 10-foot lengths. This had to be done to a whole series of piers, and took several years to complete.

In another case nearby, a slight sinkage in a short length of a long viaduct gave the maintenance staff a considerable amount of concern because the foundations were obviously sinking slightly, and it was feared that the same trouble might spread to a greater length of the structure. Investigation showed that the ground consisted of 5 to 8 feet of loam, on about 4 feet of good ballast overlying 40 to 50 feet of fine sand. It was not known whether the viaduct was on piles, which in any case might have deteriorated, or if there had been local relaxing of the sub-strata, since the general level of groundwater was known to have fallen gradually over a period of years. After further investigation it was considered that the subsidence was brought about by the large weight of metal and other commodities stored under one of the arches by a tenant. The weight was estimated at about 400 tons, part of which was on the footings of the piers. The compression of the ground on which the viaduct and the stored load rested showed up in the elevation of the structure.

The scheme for relieving the foundations is shown in *Figs 11*. It was successfully completed without the imposition of a speed restriction on traffic over the viaduct.

Another interesting problem was encountered at Hurstbourne, between

Figs 11



SCHEME FOR RELIEVING VIADUCT FOUNDATION OVER LONDON BRIDGE STATION

Basingstoke and Andover. The Bourne valley is crossed by a viaduct with a maximum height of 65 feet, consisting of nine semi-circular brick arches each of 46 feet span, carrying two tracks. The permanent way was originally supported on these by five longitudinal jack arches springing from walls built up from the arch rings and piers. These were of relatively light construction and had gradually deteriorated since they were constructed in 1854. Extensive repairs therefore became necessary.

The simplest solution would have been to replace the walls and jack arches by concrete, but tests indicated that this increase in weight at the high level would have been undesirable, and would probably have caused the viaduct to sway under fast-moving heavy trains. Foundation trouble might also have resulted.

The scheme adopted therefore consisted of :—

(*a*) Replacing the jack arches by lightweight mass concrete.
(*b*) Laying a continuous waterproofed 9-inch-thick concrete slab throughout the length of the viaduct, laid to falls for drainage purposes.
(*c*) Lowering the track by 2 feet to prevent any increase in the final dead load.

The mass concrete was of particular interest. Its aggregate was of fine and coarse foamed slag, and when it had set, its weight was only 60 per cent of that of normal concrete.

It was fortunate that the track could be lowered 2 feet, since this enabled more than 1,000 tons of ballast to be removed and so to a large extent the extra weight involved in the repairs was balanced.

Single-line working at a speed of 15 miles per hour was arranged, first on one road and then on the other, so that half the width of the viaduct could be dealt with at one time.

In order that the stability of the structure could be assured while only half the width of the viaduct was subject to live load, it was necessary to fill the spandral cavities before any appreciable amount of ballast was removed. Timbered excavations were therefore sunk over each pier for the half width of viaduct of which possession had been obtained, and the jack arches broken through. Concrete was then packed in so that for the whole length of the viaduct for the half-width being treated the space between the ballast and the rings of the main arches was more or less solid.

In order to support the track which was open to traffic while the ballast of the other road was removed, a wall consisting of bags filled with sand and cement was built in a trench alongside the sleeper ends. When the brick deck over the spandrel arches was exposed, it was found that the mortar had perished, and there was no difficulty in removing it. The placing of concrete then continued and was followed by the reinforced-concrete slab, waterproofing, and track ballast. When the permanent

way was completed, the line was opened to traffic and the other road treated in the same way.

Some details of the scheme are shown in *Figs 12* (p. 126).

The intermediate piers of Charing Cross Bridge consist of a series of cylinders, about 7 feet in diameter. They are made up of hollow iron sections, filled with what is suspected to be loose rubble. Some of the sections have cracked from top to bottom.

To prevent conditions getting worse, steel straps were placed round the faulty cylinders, as shown in *Figs 13* and *14* (facing p. 121).

The Port of London Authority, however, required that projections from the original work should be as small as possible, in order that river craft, both small and large, could pass without danger of damage. The straps were therefore joined by welding and the method adopted is as shown in *Fig. 14.* The lower part shows a strap tightened up and held in position ready for welding. The upper part shows the finished joint, with the outstanding legs of the temporary angles burnt off.

Owing to the increase in weight which has to be carried by viaducts and arches, there are many cases in which the spandrel walls have started to give, often tending to slide along the top of the arch. In many instances the movement has been stopped by putting steel ties between the opposite walls. It is, however, essential to cover their whole length with some compound to prevent corrosion, because they are not easily inspected and failure could occur with little warning.

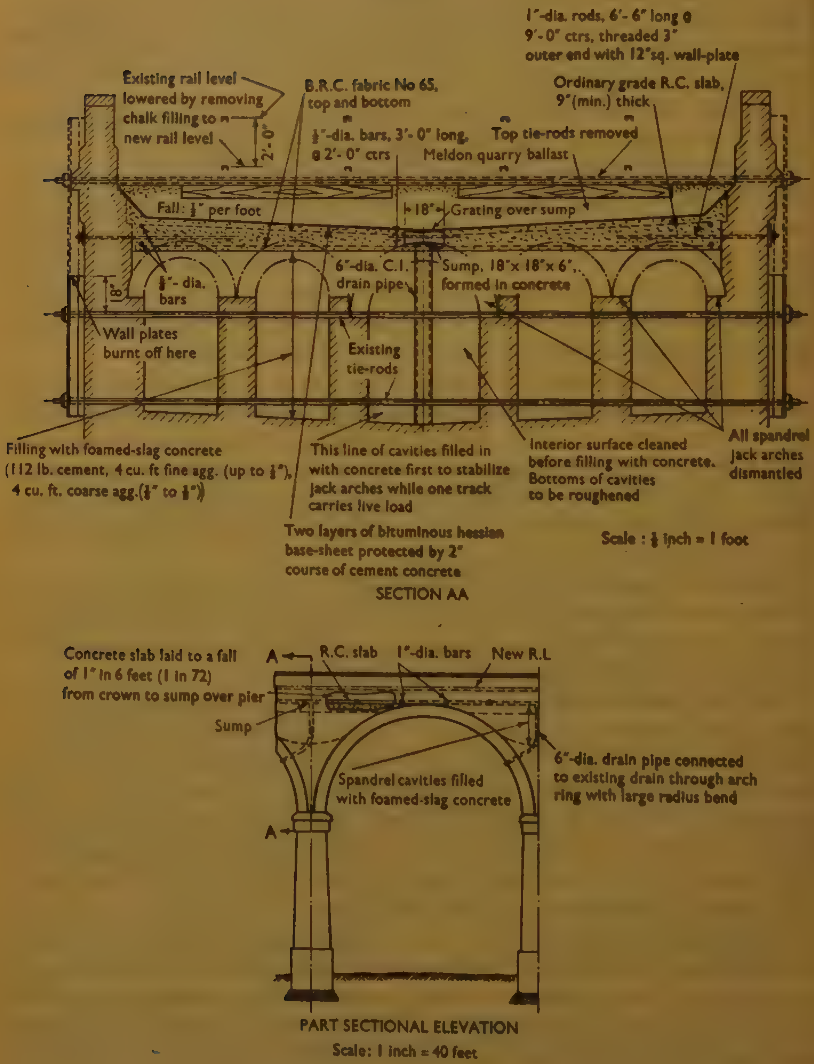
Where the spandrel wall tends to tip outwards this same method can be used, or alternatively the walls can be thickened. The latter method was used at Oxted Viaduct, where it was also desired to rebuild the parapet walls in order to give greater clearance from traffic.

On this same viaduct, other trouble was being experienced. In two arches, the lower two rings of the four rings of brickwork had fallen away slightly from the others, the cause of which was not ascertained with any certainty. The two rings were therefore taken out and rebuilt. This latter type of work is one which is not unusual in arch maintenance. Drummy brickwork is often a sign of coming trouble, and if reasonably early action is not taken, more extensive faults will develop. Cases have been known of part of an arch ring falling out.

A big problem is presented by some of the larger station roofs. That at New Street, Birmingham, has been taken down; that at Cannon Street will probably be taken down in a few years' time, and schemes are on foot for similar action at other stations. The modern tendency is to build lower roofs of the umbrella type over the platforms, or canopies projecting from the station buildings, except over terminal concourses.

Some of the bigger roofs must, however, be maintained for many more years. One such is over the so-called "Central Section" or the former London, Brighton and South Coast side of London Bridge Station. The greater part of the roof is in very good condition, but some of the arch

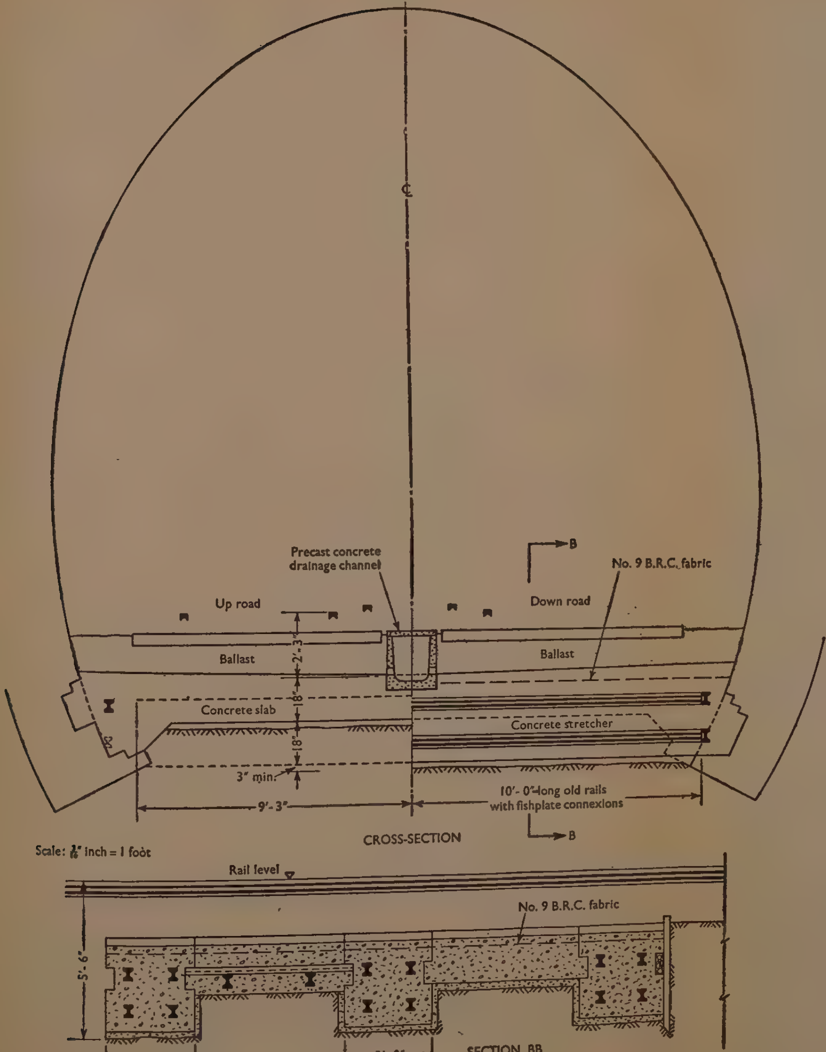
Figs 12



REPAIRS TO HURSTBOURNE VIADUCT

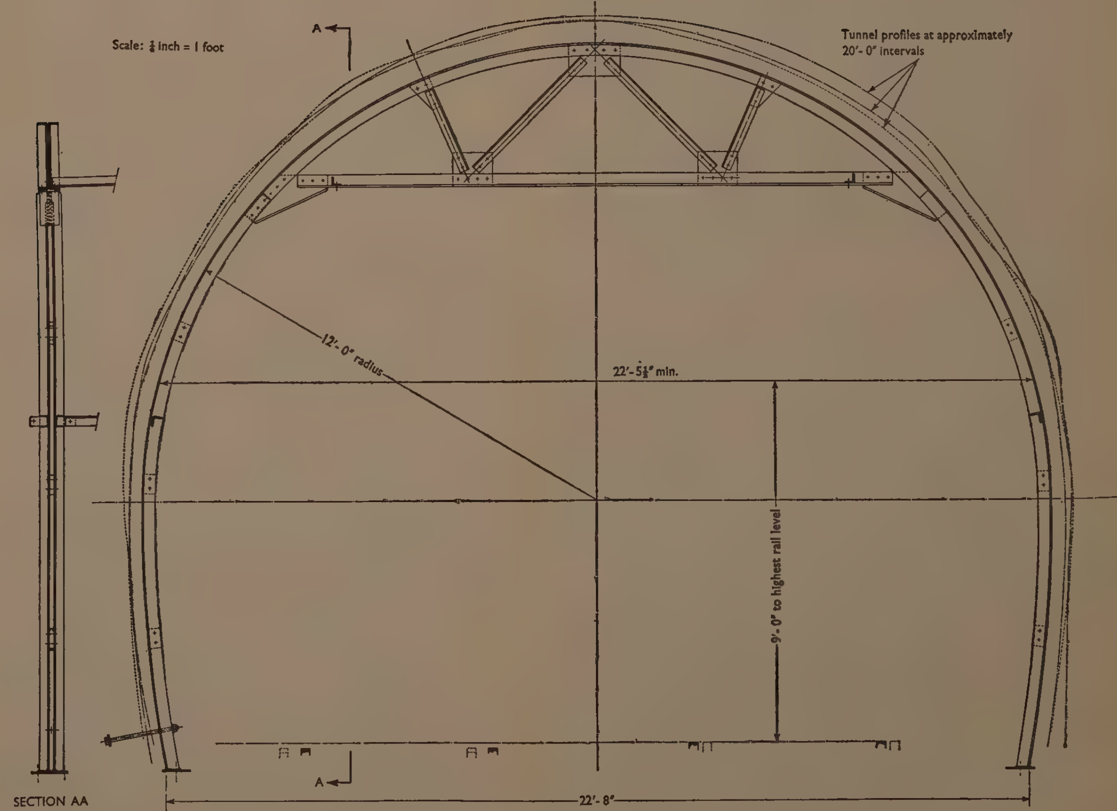
SOME MAJOR PROBLEMS IN RAILWAY CIVIL ENGINEERING MAINTENANCE

FIGS 3



STRENGTHENING OF HASTINGS (WARRIOR SQUARE) TUNNEL

FIGS 4



TYPICAL STEEL CENTERING USED ON SOUTHERN REGION

A. H. CANTRELL

spans at the ramp end of the platforms have corroded to such an extent that it was considered unsafe to replace the glazing after the war. They are therefore being reconstructed.

In this case, in order not to detract from the general appearance of the roof, it is necessary to adopt a design similar to that which is to be replaced. The part of the station it covers cannot be closed, for traffic reasons, and suitable gantries or staging for assembling or erecting a steel roof are not practicable. The present arches are not sufficiently strong to haul up heavy material and therefore it has been decided to reconstruct part of the roof in aluminium, which can be done from a relatively light gantry, constructed so as not to interfere with traffic. This is the biggest work in aluminium so far authorized on British Railways, and should prove of great interest.

Conclusion

The Author has endeavoured to show the great variation of tasks which have to be undertaken by those responsible for the maintenance of a railway. He is aware, however, that there are many others which are of interest, but space is not sufficient for them to be included.

His intention has been to give main details of some works in which special features were involved, hoping that in the discussion members will be able to give examples which have been treated in a different way.

Acknowledgements

The Author wishes to record his thanks to Mr F. E. Campion, M.I.C.E., Civil Engineer of the Southern Region of British Railways, for permission to refer to the various works mentioned in the Paper, and to various members of his staff for help in checking information given.

The Paper is accompanied by nine photographs and six sheets of drawings, from which the half-tone page plates, the folding Plate, and the Figures in the text have been prepared.

Discussion

The Author introduced the Paper with the aid of a series of lantern slides.

The Chairman, in proposing a vote of thanks to the Author, observed that during his travels on British Railways he had often heard young engineers complain that they had to go out in terrible weather, at all hours of the night, to deal with problems which arose on the railways, and that they envied those who were in business or in other departments

of the railways and who could sleep soundly in bed at nights and did not dream about defective structures. The only answer which he could give was that if they did not want to be burdened with those problems they should not have become engineers; and, however, troublesome such problems might be—and he, like other railway engineers, had had his share of them—life would be very uninteresting without them. In the Introduction to his Paper, the Author had said that " Civil engineering maintenance does not sound a very interesting subject. . . ." That view might be held, but all railway engineers knew that it was in fact very interesting, and the Author had very ably proved that that was so.

The Author had stated that the District Engineer should regularly walk all the routes under his control, and that " Contact between the engineer and his men is invaluable." The Chairman expressed the opinion, particularly for the benefit of the younger engineers present, that that was essential. Human contacts were necessary in all walks of life and were probably more important in the Civil Engineer's department of a railway or of any other undertaking than anywhere else. When he had been a District Engineer, he had kept a map on which he marked which parts of his district he had walked, and by the end of the year he had expected to see the whole of the map filled in. As the years went by he had come to know well all the Inspectors and Gangers, and even some of the lengthmen.

Mr F. E. Campion congratulated the Author on the wide variety of works which he had been able to describe in a Paper of limited length. There were necessarily, as the Author had said, many problems which had had to be omitted. All those who had had a lengthy experience of railway work would agree that such a Paper as had been presented was most valuable not only to District Engineers, in giving them an opportunity to exchange ideas, but also in suggesting essential features for new works and design, since design was of very little value if it was not properly related to subsequent maintenance problems.

The Author had referred to the " blanketing " of track, and it would be interesting to know of his experience of grouting weak formations. Mr Campion's experience in that respect was very limited ; the efforts which had been made in his Region had been the reverse of successful, and he was not at all sure that conditions had not been worse after the grouting had been done than they had been before.

Would the Author enlarge on the advisability of taking soundings round the foundations of bridges crossing rivers or streams. On Mr Campion's Region, in one instance where the foundations of a bridge had been giving trouble, soundings had disclosed a certain amount of scour in a small stream, but apparently nothing excessive. Fortunately it was a bridge of several spans, and it had been possible to divert the stream during the summer from one span to another, leaving the original bed dry. That had

disclosed a most alarming state of affairs, which had not been detected by ordinary means, but which would no doubt have eventually led to the failure of the structure.

Another question on which the Author's views would be of value was that of the periodical and systematic examination of structures, and the benefit of such an examination in detecting incipient failure and so preventing the need for some of the major works described in the Paper. In dealing with that, would the Author also give his opinion on the value of check dimensions on masonry and brick structures, with particular reference to tunnels in clay, and especially those tunnels where there was no invert.

Mr J. Taylor Thompson observed that there would be general agreement that the Paper illustrated very well the difference between railway civil engineering and what might be called normal civil engineering, that is to say, new work. In "normal" civil engineering—the building of bridges, dams, marine work, and so on—the interference experienced, if any, was largely from nature itself, whereas in railway civil engineering the most important feature was the need to avoid interference with traffic on the line. That seemed to stand out very clearly from the Paper as a distinguishing mark of railway civil engineering, which was always concerned with how to decide on the type of work, and to design and carry it out with the least possible interference with normal traffic running. That was a very important distinction between the two types of engineering. The big problem of railway civil engineering could be put in one sentence: it was so to maintain and renew the railway and its structures as to ensure a satisfactory standard of maintenance year in and year out at the minimum cost and with the minimum interference with traffic. The Author had shown that that was a major problem.

Mr Taylor Thompson agreed with the Author that the problem of track maintenance was basically one of organization. A stage had been reached in railway history which might almost be called the parting of the ways. Flat-bottom rails were now gradually taking the place of the old bullhead type, and there was now a considerable mileage of them; combined with that there was a shortage of man-power and difficulty in getting man-power of the right quality. Those two facts provided good cause for thinking afresh about the problem of how in future to maintain railway track in a satisfactory condition at the lowest cost and in the most efficient way. To justify the use of such heavy track it was necessary to reduce the overall cost and, in particular, to reduce the man-power required to deal with it; that would involve, it would probably be agreed, a certain amount of mechanization, with mechanized gangs covering a considerable length of railway, combined with small gangs doing the normal inspection and patrol work.

The portions of the Paper dealing with formation, slips, and scour might be summed up by saying that water was the root of all evil in railway maintenance work, whether in relation to the track or anything else. It

9

was obvious, therefore, that too much attention could not be given to drainage. In years gone by there had been a tendency to leave drainage to the Inspector, or perhaps to even lower grades, but to-day it was essential that drainage should be properly designed and carried out, so that the money necessary was effectively spent. Money spent on good drainage was money very well spent indeed.

The Author had referred, when dealing with scour, to the use of wire cages filled with stone—gabions—and seemed to suggest that their use had not been adopted in Great Britain. Mr Taylor Thompson could remember that idea being used 20 years ago to deal with river scour, and being found very effective indeed. He agreed with the Author that a wire-mesh cylinder filled with large stones was an excellent means of dealing with river scour.

Mr Taylor Thompson could confirm that many present-day maintenance problems would have been reduced if a former generation of railway engineers had built even more masonry structures than they had done. Some of those large masonry structures were carrying to-day, without any sign of distress, far greater loads than they had had to carry at the time when they were built, and they were still in excellent condition. It was interesting to find that the Swiss engineers, even to-day, were replacing steel and wrought-iron structures by masonry ones, taking great care that they should be beautifully designed, but also having in mind that maintenance was small compared with that necessary for metal structures. In replacing metal viaducts, the Swiss engineers had in many cases taken long-distance photographs of the site, and then provided a masonry structure which harmonized with and, if anything, improved the beauty of the scenery. Some of their masonry work was really a delight. In Great Britain it was almost impracticable to replace metal bridges by masonry ones because of the interference with traffic, but were it possible to do so, great benefit would be derived.

The Author in his oral introduction had referred more than once to the effect of age on structures, but Mr Taylor Thompson thought that the most important factor that affected metal structures was the quality and the standard of maintenance, rather than age as age. There were tunnels on British Railways which were 100 years old and still in first-class condition, and other tunnels 50 years old which were in poor condition. It was largely a question of the surrounding circumstances—the materials through which the tunnel was cut and the standard of maintenance. So far as metal structures were concerned, he could not recall a bridge being rebuilt because of fatigue, although "fatigue" was a very popular word to-day. Tests which had been made on more than one old bridge, comparing the stressed part with the unstressed, had shown that age and use had had very little effect on the quality of the material. It would appear that fatigue—in other words, mere age, and the number of times a structure had been stressed—was not as important as was sometimes thought.

Railway engineers of 125 years ago had had to design those structures without that experience of maintenance conditions which was now available. One lesson which engineers could learn from the Paper and from everyday experience was that, in renewing structures or in building new railways, they should so design and build them that their successors would not have the maintenance troubles which they themselves inherited from their predecessors.

Mr J. D. West, referring to some recent maintenance work on the Eastern Region, stated that at about 11.50 a.m. on 24 September, 1951, a part of the Sheffield Victoria Station roof had collapsed, after only a brief warning of impending failure by fragments of the cladding falling away. Much credit was due to the district staffs of the Civil Engineering and Traffic Departments for the precautions taken by them, which had been such that the roof fell into an empty station. There had been about 20 minutes' warning in which to take the necessary action, that was, to clear the platforms (using the loud-speakers on the station) and to seal-off the various entrances and exits.

Four trusses had been involved, and failure had been traced to an extensive but concealed fracture in the cast-iron apex-block of one of the trusses. The sudden development of the fault had severed the casting and the truss then being an incomplete frame had failed. The concealed fracture had been found to extend for about 90 per cent of the casting. It was not known how it occurred, but it had evidently been of long standing, and it was thought that possibly blast from bombs which fell near that end of the roof during the war might have started it off.

By far the greater part of the roof had remained standing, but, of course, had been suspect. A comprehensive examination had been conducted from long ladders operated from a tunnel-examination vehicle, and a further truss taken down for detailed examination of the joints. That had revealed some concealed faults in the cast-iron members, and the decision to demolish the whole of the roof had been taken. The concealed faults that had been found after complete demolition were in members other than the apex-blocks; the only apex-block found to be fractured had been the one in the truss which failed.

The demolition work had been conducted from two travelling gantries constructed for the purpose, the work being completed without incident. In order to restore the station to normal working, it had been necessary to provide covering to the platforms as quickly as possible, and that had been done by using asbestos-cement cladding on steel scaffolding; by Christmas a very substantial area of covering had been provided.

At first sight there would appear to be almost unlimited possibilities in preparing the design for the permanent roof, but that had not proved to be the case. The station walls had been in excellent condition and it had been extremely difficult to disregard their value as supports for the new roof. Moreover, the station was supported on arches, and foundation

difficulties had been at once apparent when alternative methods of support were considered. In other directions, an irregular building line presented some further problems. It had seemed that a high roof of the overall type, with its inherent faults, was possibly the only answer, but in a station mostly used by steam trains that had been considered to be an extremely undesirable solution and a retrograde step from a maintenance aspect.

The design finally adopted provided for a light glazed roof on welded steel framing over each platform. The units were suspended from prestressed concrete beams, at approximately 25-foot centres, which were seated on the existing station walls. The suspension rods were of steel; they were, of course, the one potential weakness in the design from a maintenance point of view, and they would be sheathed for protection against corrosion with inert materials. The roofs would be anchored rigidly to the walls at the mid-points, and elsewhere by links to permit expansion, at the same time resisting horizontal wind forces. There would be all the advantages of a low-level roof so far as glass-cleaning and maintenance were concerned, and there would be freedom from obstruction at platform level which was associated with the original high-level roof. In addition, the prestressed concrete beams would be available for the support of overhead traction wires, obviating the necessity to erect independent masts. That was an important point, because it had been intended, had the old roof not been removed, to have masts standing on the platforms to carry the three lines of conductor wires.

With regard to the suspended portion of the roof, and from an architectural point of view, equal importance had been given to the transverse members and the purlins, so that the roof was virtually divided into panels when viewed from the inside. The soffits of the ribs and the purlins were in one plane, and the joints would be welded.

Mr E. K. Bridge remarked that not the least interesting aspect of maintenance was finding out what had caused the trouble. It was very little use patching a structure which had failed without finding out what had caused the failure. Sometimes that was quite an interesting quest and, strange though it might seem, it was often masonry structures which gave the most trouble. The Author had mentioned Hurstbourne Viaduct. That had been interesting from that point of view, because it had been necessary to find out why the viaduct was swaying. The spandrel walls had been shaky and some strengthening was required, but the problem had been to know whether to add weight or to take it off. The addition of weight would have altered the period of vibration of the viaduct, as would the removal of weight. The problem had been to decide which course to adopt to avoid making the natural period of the viaduct coincide with the oscillations produced by goods trains travelling at 40 miles per hour. Collapse might have resulted if they had chosen the wrong way.

One track had been loaded successively with engines of different weights, and the oscillations from a test goods train had been measured by oscillo-

graphs on the viaduct and deflectometers attached to wires from the side of the viaduct to the ground. Those clearly indicated that increased weight would increase the oscillation, and the reduction of the top weight by the repairs described tuned the viaduct away from the period of oscillation caused by the goods trains. That had been successful.

In the case of another structure, the Tavy Viaduct, the cast-iron cylinders forming the piers had been continually developing vertical cracks which had had to be repaired with steel bands. Investigation indicated that stalagmites had been forming on the lower portions of the stone filling, apparently caused by rainwater draining in at the top of the cylinders. The tops of the latter had been made watertight with bitumen, and little or no further cracking had been experienced since then.

Two bridges on the Ringwood line had been under serious consideration for renewal at about £30,000 each, since they were required for heavy traffic, and continuous settlement up to 15 inches had occurred in a number of the piles. Investigation had revealed that, of the three piles in each pier, the centre one had not sunk so much as the outer piles, and probes could not be pushed down into the bed of the river more than an inch or two beyond the bottom of the surface peat. Since that had indicated that there was probably fine sand there, which had a high bearing value unless "pumping" of the foundation took place, tests had been carried out and revealed that the viaduct had been tilting towards the Up and Down sides under the passage of trains on those tracks. By jacking up the bridge and increasing the packings on the outer piles only, the rocking about the centre pile had been eliminated and the structures made available to the heavy traffic without reconstruction, at a cost of about £215 for both bridges. Little or no further settlement had since taken place.

With so many wrought-iron bridges on the railways, the successful technique evolved for the repair of the bridge over the River Thames at Staines, described in the Paper, opened up a valuable field for economy in wrought-iron bridge repairs. Wrought iron was generally accepted as a very difficult material to weld. All the many welds on Staines Bridge had been carefully checked and in no case had a crack occurred. The welding had been done without stopping the traffic; the welders had stopped work when a train crossed the bridge and started up again thereafter.

Mr Bridge agreed with the Author that buttresses were generally of little use in supporting defective masonry structures, since, unless their foundations could be pre-loaded to give the same ground compression as that existing in the main structure, the buttresses tended to settle and drag out the wall which they were intended to support.

The question of fatigue had been raised by Mr Taylor Thompson. Although in main girders the number of really heavy loads which could cause fatigue was hardly enough even in 100 years to damage a structure, sight must not be lost of the fact that corrosion fatigue could damage a

structure with a smaller number of applications at a very much lower stress. It was corrosion fatigue which was the danger.

The Author had mentioned bridge composition as a protection for the upper surfaces of the bottom flange plates of girders, but would agree that in many cases that gradually worked loose from the steelwork and held moisture against the steelwork, causing it to corrode. It was probably better to leave the steel open for painting and maintenance.

Many brick structures on the railways and elsewhere had been built using lime-mortar, from which the lime had washed out over a long period of years. Since the sand in many cases was too fine to permit cement-grouting of the joints, experiments had been carried out on the possibility of chemical consolidation of the mortar, but the difficulties of high cost and low strength of the mortar produced had not yet been overcome. One of the problems which would have to be faced, however, was that of finding some method of getting new mortar into old viaducts.

Mr F. J. J. Prior observed that the Author had shown what an immense task lay before the engineers of British Railways, not, as Mr Taylor Thompson had remarked, on account of the age of any particular structure, but because of the very large number of structures which were ageing and becoming due for maintenance and repair at the same time. The Author had referred, under the heading "Formation and Foundations," to permanent way drainage. Mr Prior thought that deep formation digging had been largely brought about by the lack, or failure, of existing drains. It might happen that drains which appeared to be functioning quite well and to be maintained in good condition by the gang, with catch-pits kept clear and pipes running freely, still did not take the drainage from the track ; in his view, a programme of renewal of the filter material over the drains would pay very well and, with deep re-ballasting of 9 to 12 inches, eventually avoid the necessity for deep digging of 3 feet 6 inches to 4 feet.

In tunnels there was the usual problem of spalling brickwork, but another feature, which the Author did not appear to mention, was the softening of mortar, which was very unusual. In one instance which Mr Prior had in mind, the mortar in the tunnel had perished throughout to 1 inch in depth, and in places 2½ inches. A sample of the water from that tunnel had been sent to the Cement Marketing Board, who made an analysis and reported that they could not offer advice on any medium for pointing. A certain firm had suggested a mortar with a rubber base, and seemed sure that it would be successful, since it had been used in the lining of acid-holding tanks, provided that it could be made to stand in the joints of the tunnel. For the initial period it was necessary to have a dry tunnel, which, of course, was very rarely obtained. For the purposes of the test, however, it had been decided to take a compressor into the tunnel, behind the tunnel van, and blow compressed air on to the brickwork and dry it out yard by yard. That had been done quite successfully, and the mortar had been raked out to an average of 1 inch deep, made good with

2 : 1 rapid-hardening cement to ½ inch deep, and then pointed with the rubber-base cement. It became quite hard after 30 minutes. Unfortunately Mr Prior had left that district and did not know the result of the test ; it would, of course, take 2 or 3 years to find out whether it would soften under sulphur conditions. He would welcome information on that point or on any similar experience.

The Author had mentioned the opening out of girder ends which had been boxed-in with brickwork. It might be of interest to mention that in Mr Prior's district, of 100 bridges under repair during the current year, twelve were, in fact, being opened up at the girder ends. While that was being done, opportunity was taken to remodel the bearings, remodel the bedstones on grillages, and to set back the ballast walls, so that in future the bearing ends would be accessible at all times. That had been done using military trestling braced to the abutment, so that the whole work could be carried out with no restriction of speed.

Mr G. D. S. Alley said that the subject matter of the Paper was so large that naturally the Author had been able to deal with only some of the major problems which were encountered on all Regions. Since, however, the Author was known to be a great exponent of permanent way engineering, and since some of the photographs in the Paper showed longitudinal timbers on bridges, it would have been interesting had he said a little more about the troubles which such a type of construction brought to the permanent way engineer. In the days when those bridges were contructed, timber of reasonable section and of good quality could be obtained. Running from a fairly flexible sleeper-track on to a longitudinal timber often led to trouble in the way of maintenance. Would the Author say if it was the practice on the Southern Region to terminate longitudinal timbers level with the deck of the bridge or to carry them over the back of the abutment in any way.

On p. 117 of the Paper the Author had referred to the method of strengthening flanges of bridges by welding, and there was one point there which ought to be emphasized. When a portion of a flange was cut out, unless some of the remaining stress in the bridge which was left prior to welding-in additional steelwork was relieved, it was anybody's guess what that new steelwork was going to do. There were difficulties in doing that, but it could and had been done.

Figs 3, Plate 1, showed details of the strengthening of the Warrior Square Tunnel at Hastings. Mr Alley showed a slide of a tunnel (or bridge, if that description was preferred) which was 161 yards long and which had begun to give some trouble about 10 years ago ; one of the abutment walls on the Down side had started to move in. Temporary timber struts had been hastily put in position and strutted across from the piers in the centre to the abutment wall on the Up side. That had stopped the movement temporarily, or at least reduced it, and had given time for other remedial measures to be planned, consisting of putting a longitudinal

beam along the portion of the side causing most trouble, and cross-strutting.

A second slide illustrated the type of construction, using a beam made of rails bolted together and reinforced concrete for the main strut, the longitudinal beam being of a similar type. Those had been placed in short sections along the length of the tunnel, and had to be put in without unduly interfering with the timber struts, which required careful planning. That, he believed, stopped the movement. The reason he mentioned it was that that work had been carried out without such severe blocking of the track as complete inverting would have entailed. Rails as struts were not regarded with great favour by some engineers. It was understood that they were wasteful of steel, but he would emphasize that rails used in a manner such as that, or as described by the Author, lent themselves to ease of handling and ease of erection on the site.

Mr A. K. Terris said that the importance of the Paper was that it pointed to the fact that the new works of to-day were the maintenance tasks of to-morrow, and that no matter how much the works of one's predecessors might be admired, there could be no doubt that the effects of age and use were now very evident. In the future, the work of the Maintenance Engineer would increase very considerably because those agencies not only affected the structure but in many cases the ground upon which it was founded. The latter feature had been most manifest, particularly since he came to the south, where the clays in the southern part of Great Britain caused more deterioration in foundation conditions than had been his experience elsewhere.

Because of the factor of age and increasing necessity for remedial works, it was essential that a sound technical service, both at District and Headquarters levels on the Railways, should exist to deal satisfactorily with inspection, reporting, assessment of work required to be done, and the manner in which it was performed, so that manpower and materials were continually being conserved.

The Author had referred to the sand-piling method of track improvement. In the technical press 3 years ago, that method had been described as carried out at Waddesdon on the old Great Central main line. The Author had indicated that one of the features of that method was expected to be that the sand piles or " spuds " would provide a drainage system. Mr Terris did not think that he could agree on that, having regard to the experience obtained from tests carried out immediately after the work had been performed. There had been no apparent indication that there was any capillary action, or any lessening of the moisture content of the clay because of the sand pile. On the other hand, there had been certain improvements in the condition of the clay, but those seemed to result from the compaction which took place in the clay and the better average quality of the formation which resulted because of the insertion of the sand-pile. The track in question, however, had passed under the jurisdiction of the

London Midland Region some time ago, and perhaps they could give more exact information; the last time he had asked about it the report had not been unsatisfactory. At Waddesdon there had been a limestone band, 4 to 6 feet down, which had appeared to provide a very suitable place at which to try out that form of piling, with the expectation that the water would gradually seep away; the method, if successful, allowed work to be done with single-line possession and at a much reduced cost compared with the more general method of blanketing.

On the Eastern Region, a feature of blanketing work had been the insertion of a waterproof layer on the top of a 3-inch thickness of sand on the top of the formation. He appreciated that there were two schools of thought on that matter. Although that method had been experienced for only 4 years, a test had recently been carried out and a trial hole made, and it had been obvious that the waterproof sheeting was in splendid condition. It had not seemed to be undulating to any extent and it still maintained a straight fall to the cess and therefore provided what was required, namely, a direct means of shedding water away from the surface of the clay. The new exposed surface of the clay was in a satisfactory condition and there were no undue signs of sweating or becoming soft.

On the Eastern Region there was no convenient source of stone dust, and the material used was sand. According to the best practice, attempts had been made to find a fine-sand filter which equated to the kind of clay with which they were dealing, but it should be mentioned that the possibility of doing that seemed to be limited, because of the types of clay which were encountered where the particle-size of the clay had been very small indeed; to equate the filter to the particle made it necessary to obtain a sand which was extremely fine and which in practice would be far too fluid for the purpose. Sharp sand, capable of compaction, was now again being used.

The Author had referred to the troubles resulting from the effects of mineral subsidence, and Mr Terris showed a number of slides illustrating those effects. The first showed the portal of a tunnel which until a few months ago had been in active service, and he suggested that it showed the limits of human endurance of Engineers, District Engineers, and their men in matters of maintenance. The photograph showed a double row of ribs, all heavily lagged, bracing on top, and struts below. It had been kept going by those means for 20 years, and dispensed with only when economics could not justify its retention. The second slide showed a viaduct adjacent to that tunnel and again represented what it was possible to do, not by maintenance so much as temporary support of a structure to maintain traffic running.

The third slide represented a cross-section of an embankment in the Fen country in Cambridgeshire. It was thought by those who had made a study of the matter that the line of the peat had been originally as shown on the diagram. There were three factors in the problem of the maintenance

of embankments or formations in the peat country. One was the shrinkage of the Fen because of the drainage operations which were being carried out in that part of the country. Another factor was the erosion of the soil by wind, which he understood was quite appreciable, and a third was some biological action which was taking place and which still further reduced the level of the fens. It was estimated that the Fen was going down at the rate of 1½ inch a year. It was clear that in the Fen country there were those earthwork problems affecting railway structures, the existence of which might not be appreciated.

Another slide showed the condition of a house adjacent to the lineside in the Fen area. Within the area of that building, strange to say, the peat had remained at a higher level than outside. That feature also applied under the railway as shown in the diagram. A further slide showed the effect of the shrinkage which took place on structures generally, with telegraph poles sloping, and fences and a platelayers' hut affected.

A further slide showed an abutment and wing wall which had failed because of the weakness of the foundations. The method of strengthening was illustrated, reinforced-concrete facework being built into the front of the abutment and founded by underpinning the existing abutments. The foundation was made of sulphate-resisting cement, because in Southern Essex concrete was seriously affected by sulphates in the clay.

The last slide depicted what Mr Terris described as the demolition of a redundant asset (the blowing up of a bridge over the railway); it typified one aspect of the work of those who were concerned with the maintenance of railway works. It was no use spending money on something of no further service, no matter how much sentimental value might be entailed.

Mr J. D. Watson expressed particular interest in the Author's remarks about gabions, which Mr Watson regarded as a very useful item of stores to have for emergency work. Use had been made of them on a job which had been carried out by the Royal Engineers in Italy in December 1943, on a bridge, between Benevento and Foggia, which had been dropped at one end. The lower end of a 50-metre span, which was in the river bed, had been very badly damaged, but the rest of the steelwork had been serviceable. They had decided to burn off the damaged steelwork, raise the remaining part of the big span, support it on a temporary pier, and make up the piece which had been lost by an approach span. Only a very short time had been available for the work, and it had been necessary to found the temporary pier on a timber raft of 12-inch-by-12-inch timbers.

The river was a mountain stream with almost no water in it at the time, but, having had a certain amount of experience of similar rivers in India, Mr Watson had realized that if there was much rainfall, the water would come down the river in a terrific spate and there was every chance of the temporary pier being washed away. It was accordingly decided to build a groyne (or, as they would have called it in India, a guide bund) of gabions, a number of those having fortunately been found in stores in Naples.

A gabion, when in store, was like one of those cardboard containers which could be folded flat; the four sides were folded down and delivered to the site in that form.

Italian stonemasons, in the river bed, had put the gabions where they were wanted and lifted the sides and wired them up; a large number of Italian ladies had carried boulders on their heads and dumped them into the gabion. The gabion in section was 1 metre by 1 metre; it was either 2 metres or 3 metres long and it was like a very-large-size building block. They were built, rather like brickwork, in headers and stretchers. The groyne, or guide bund, which had been constructed looked quite a formidable and workmanlike job. There were two layers of gabions, one of headers and one of stretchers, so that the groyne was 2 metres high and, he thought, 3 metres wide. The girder had been raised by a system of wire tackles from an overhead gantry, the power being supplied by a ocomotive going along the track. Sleeper cribs had been built as fast as possible underneath.

On New Year's Eve it had rained very hard all through the night, but work had continued. It had rained all through New Year's Day, and by the night of 1st January it was apparent that the work was going to have a very severe test. The river came down in flood, but the gabion groyne, which had been designed to deflect the stream away from the temporary pier, did its job, and, though breached at one point, it lasted long enough to save the temporary pier, which had practically no foundations but was just standing on the river bed on the raft. The following day the spans had had to be adjusted a little by jacking up and packing, but it had been possible to pass trains over the bridge.

He had then been called away to do another job, but he had left instructions for improving the gabion protection, and he believed that the bridge continued afterwards in serviceable condition. The sequel was perhaps of sufficient interest to mention. A Railway Bridging Section R.E. had made drawings of the missing steelwork which had been burned off. That steelwork had been fabricated by an Italian firm in Naples under Royal Engineers supervision, brought to the site and riveted on to the span—in doing the burning, care had been taken not to damage the gussets so that that could be done. Finally the span, restored to its original length, had been landed on its bearings and the temporary pier dismantled.

Mr A. H. Toms strongly supported the remarks of previous speakers to the effect that the design of new structures must take account of such factors as secondary stresses and racking stresses which had not been considered in the design of many old structures, which had suffered in consequence by loosening of rivets and overstrain of bracing and other members.

A speaker had referred to the strengthening of deteriorated mortar in structures such as tunnels, and Mr Toms mentioned that prior to nationalization an investigation had been started on the Southern Railway which

had been carried on since by a chemist, now with the Railway Research Department. Certain very encouraging results had been obtained, but Mr Toms was not in a position to disclose them since he was no longer responsible for the investigation. He believed he was right in saying, however, that it looked like being a costly business.

With regard to the permanence of masonry structures, on a brick-arch viaduct at Wivelsfield, in Sussex, a very large thickness of the spandrel split right away to a depth of up to 18 inches owing to frost action resulting from water penetration through the formation of that viaduct, so that he did not think it could be said that masonry structures were free from maintenance.

He had had to make an investigation of a steel and wrought-iron roof at Devonport station, which had cast-iron bulb-plate gussets clamped round a bulb T-section leaving a space of about ½ inch of bolt where it passed through the gussets into the T. Everything outside looked satisfactory, but almost complete destruction of the obscured parts of the bolts had occurred, owing to the sulphurous atmosphere. On that and other accounts, the roof had been condemned at once.

With regard to the use of asbestos-cement sheets, some attempts were being made to introduce a super-sulphate cement into the composition. Tests had been made at the Wimbledon chemical laboratory (now part of the R.E. Research Department) which showed that that composition would have a much increased resistance to sulphur deterioration, which could destroy an ordinary Portland-cement-asbestos sheet on the top of a locomotive-shed smoke outlet in about 5 years.

On Mr Prior's question about the avoidance of deep digging, Mr Toms was wholly in agreement on the importance of drainage but, so far as he could see, it would not obviate deep digging on weak formations. Much more research work, however, required to be done on blanketing methods before it could be said that all the problems had been solved.

On the subject of the protection of the underside of bridges and other structures against locomotive blast, he was satisfied, from investigations on the Southern Region, that it was not possible to find a substitute for a separate smoke plate of some material which could be much more resistant than any painted surface of steelwork or concrete.

The Author, in reply, referred to the Chairman's opening remarks and said that it was true that young engineers, when starting on the railway, did not fancy having to go out on a Saturday night, but once they had been persuaded to do so they very soon realized the value of it. Most engineers who had passed through Divisions or Districts had had a good deal of Saturday night work and they would probably all agree that on the whole it was very interesting. It was useless for someone to sit in an armchair in an office and explain how a job should be carried out if he had never been out at night himself and seen it done.

With regard to walking the District, the Author used to have a map on

which each day he marked the part over which he had done one of his routine walks. He looked at the map frequently and was either pleased with the growing amount that had been coloured in or displeased when it had come to a stop because of other jobs. It was very important to walk regularly, and the only way to plan it was by means of a map.

Mr Campion had asked for information on blanketing by means of grouting. The Author had not had much experience of it, but some years previously in the District where he then was, he had been asked to carry out a stretch, mainly from the point of view of trying it out and seeing what happened. He had not been at all keen on doing it. He had seen American Papers which explained how efficient grouting was, but every scheme described in detail had been in an embankment, and the articles, while they finished by saying that the process was equally successful in a cutting, had not given a single example. He had had his doubts about it, because he could not imagine grout percolating in sloppy clay, but if it were successful it would be very much cheaper than the usual methods of blanketing, and therefore it had been decided that it had to be tried. The experts had said that they could direct the grout wherever it was wanted, but in fact the result had been the blockage of all the drains. Some had been replaced with difficulty, but it had not been possible to get the complete system operating again. The drains being partially blocked, any water which had come down could not get away even as well as it had done previously, and the condition of that track had been worse afterwards than before, so that no more grouting of clay subsoil in cuttings had been tried.

The periodical examination of structures was very important, and on the Author's Region there had been for many years a definite system by which steel structures were thoroughly examined once every 2 years, and masonry and brick structures every 3 years. The reports went to the District Engineer, and a decision was made on whether repairs were required or not. He was not in favour of having someone who might be called a "structures examiner." It was necessary for a steelwork man to examine steel structures, a brickwork or masonry man to examine brick or masonry structures, and a timber examiner for timber. One examiner could not be expected to deal with all three types and be expert in spotting defects in each. Everyone would agree with the need for periodical examination, but there might be differences of opinion about who should do it, though he himself had no doubt at all.

A short time previously at a meeting of the Assistant Engineers' Association, there had been a discussion on soundings and the general conclusion had been that whenever a bridge was being examined soundings should be taken if it passed over water. He thought that that was reasonable. It was not easy to take soundings at some time apart from the examination, because it was likely to be forgotten, whereas if the examiner had to go to the bridge and produce a complete report soundings would be included. They should be a routine matter in bridge examinations.

Mr Taylor Thompson had said that water was the root of all evil. The Author agreed. A great deal of railway trouble arose either from flowing water or from water mixed with clay. It was the water which seemed to be at the root of it. That led to the point made by two or three speakers, that drainage problems were almost a first priority. It was no use expecting to keep a good formation unless the water was prevented from remaining under it.

He was glad to hear Mr Taylor Thompson say that gabions had been used. They had great value, and since writing the Paper he had found that the Admiralty had also used them. A case quoted to him was Scapa Flow, where the Admiralty put down a great many of them to form a breakwater. He had been particularly interested to hear that, because he had not used them in salt water and had a feeling that possibly the cage would rust away fairly quickly and become useless. He had been told, however, that those at Scapa Flow had been in use for 11 years and there was no appreciable deterioration.

Mr Taylor Thompson had also mentioned that structures were not necessarily becoming age-expired. The Author agreed that more efficient or more regular maintenance could have extended the life of many structures very considerably ; at the same time he thought that a limit did come —he would not like to say after how many years—for most of those structures. Steelwork gave trouble at the joints. Rivets became loose and patching had to be done. With increased loads steelwork suffered to a certain extent not, strictly speaking, through age, but because after it had been in use for many years and was bound to have deteriorated to a certain extent, it had to support heavier loads than had been envisaged at the time when it was designed. It was really a combination of age and maintenance.

Tunnels, of course, presented great difficulty. Some of them lasted extremely well, whilst others went very badly. Much of that might be attributed to the quality of the bricks used and also to the amount of water which came through, according to their position.

Mr West had shown some slides of the proposed new roof at Sheffield. The design was of interest, because on the Southern Region there was a proposal to do something similar at one of the stations at Bournemouth. There the roof, which spanned right across the station, had been giving a good deal of trouble in connexion with the glass and it had been decided to do away with the glazing, but to keep the main girders which went across the station, in order to suspend from them platform awnings, as shown for Sheffield.

Mr Bridge had referred to bridge compound. The Author agreed that it could be much worse to use it than not to use it, but it depended to a large extent on the actual composition of the compound. Some did not cling to the steelwork but seemed to part from it, and then water and dirt got in behind, so that the effect was, as Mr Bridge had said, to keep that

dirty material next to the steelwork and give it a chance to start corrosion. That was true, but with a really efficient bridge compound it was possible to keep that moisture away. In the instance near Battersea which was mentioned in the Paper it was not known quite how long that compound had been in position, but it was certainly 30 years or more, and the steel underneath was in excellent condition, showing that the compound had its uses provided the right kind was used.

He agreed with Mr Prior's remarks about drainage and was interested in the experiment which Mr Prior had carried out for brick pointing in tunnels. However, as had happened in many other instances, one might have a bright idea and try it out and then not know the result for many years to come. That was evidently so in the case in question. It would be of interest if in the written discussion (Correspondence) any member could give a report on that method.

Mr Alley had mentioned longitudinal timbers on bridges—an old problem. In the design of many old bridges, and especially the longer spans, the weight had been kept down by having longitudinal timbers instead of ballast. That had been so with Charing Cross Bridge, which would not take the weight of ballast. Timber was not so good as it used to be, and the Southern Region found that where track was laid on these timbers the chairs tended to eat into the timber very much more quickly than before. That was now being avoided to a very large extent, however, by putting an elm packing under the chair and on top of the timber. It projected about 1 inch all round the chair, and gave a wonderfully improved life to those timbers. With regard to hard or soft running when going from sleeper track to longitudinal timbers, the Author agreed with what had been said, but the difficulty of maintaining that change was reduced very considerably if the timbers extended, as Mr Alley suggested, beyond the end of the steelwork. On the Southern Region the usual distance was about 10 feet—a 10-foot extra length of longitudinal timber to span from the hard to the soft. It had been found to give no trouble at all, so long as the projecting ends of the timbers were kept reasonably well packed. It was necessary to persuade the ganger that that was his and not the bridge department's job.

Mr Terris had mentioned the necessity for having competent maintenance staff. The Author heartily agreed, and took the view that anyone going to an important post at Headquarters should have had District experience for many years, so that he would realize the difficulties of the Districts, which of course meant the difficulties of maintenance, and therefore could be of real assistance in easing the important problem of maintenance.

With regard to the sand "spuds," the Author remarked that it was important to penetrate as a stratum that could drain the water away. He had seen articles on the method, and sometimes people had driven down into the clay and no farther. That was asking for trouble, since it formed

a local area where water could collect and stay. When once it was full it could not take any more and the drainage aspect just disappeared.

Mr Toms had misunderstood him or Mr Taylor Thompson on the point about masonry being free from maintenance. He did not think it was, but, as Mr Taylor Thompson would probably agree, maintenance on masonry structures was much less than on steel structures.

Correspondence on the foregoing Paper is closed and no contributions, other than those already received at the Institution, can now be accepted.—SEC. I.C.E.

Paper No. 5878

"The Performance of Some Asphalt and Coated-Macadam Mixing Plant" *

by

David Bowe Waters, B.Sc., A.M.I.C.E.

(*Ordered by the Council to be published with written discussion*) †

SYNOPSIS

Measurements of performance have been made on seven asphalt and coated-macadam plants while they were engaged in normal commercial production. The results show the level of accuracy attained in each operation. Where such accuracy is unsatisfactory possible remedies are suggested. Conclusions on matters of detail are included at the end of each section of the Paper.

Two general suggestions are made :—

(1) All proportioning devices on mixing plant should be provided with a simple and direct means for checking their accuracy.

(2) Simple testing procedures should be adopted so that operators and designers can obtain an accurate assessment of the capabilities of their plant.

INTRODUCTION

THE introduction of finishing machines for laying bituminous surfacings has stimulated a demand for mixing machinery of much greater output, and the longer life now required of the surfacings requires the material to be of more constant and accurate composition. As a first step in research work directed to the improvement of mixing machinery, a series of tests have been made on several representative types of plant while they were engaged in commercial production.

Coated-Macadam Plants

Plant No. (1) An old plant having few refinements.

,, (2) A large modern plant.

,, (3) A large modern plant with automatic electric grading.

The essential features of a batch-type coated-macadam plant, typical of the two modern ones included in the investigation, are shown in *Fig. 1*.

Many commercial plants differ from the arrangement shown, but the diagram illustrates the features of the larger modern plants.

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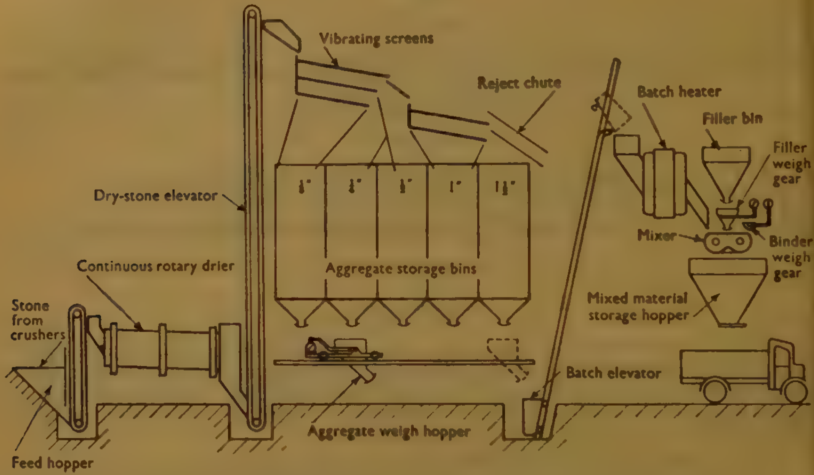
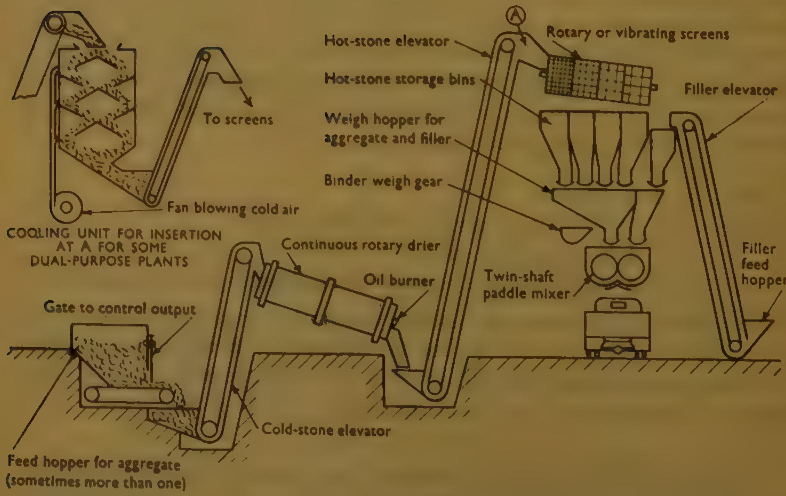
† Correspondence on this Paper should be received at the Institution by the 1st June, 1953, and will be published in Part II of the Proceedings. Contributions should be limited to about 1,200 words.—SEC. I.C.E.

Fig. 1



DIAGRAMMATIC SKETCH SHOWING THE UNITS OF A TYPICAL COATED-MACADAM PLANT

Figs 2



DIAGRAMMATIC SKETCH SHOWING THE UNITS OF A TYPICAL ASPHALT PLANT

Asphalt Plants

Plant No. (4) A mobile 20-tons-per-hour plant.

,, (5) A fixed 20-tons-per-hour plant.

,, (6) A continuous mixing plant. (The tests on this plant were made during an earlier investigation; they dealt only with variations of temperature and composition of the mixed materials.)

The essential features of a batch-type asphalt plant are shown in *Figs 2*.

Dual-purpose Plant

Plant No. (7) A dual-purpose plant.

This plant is basically an asphalt plant but the provision of the cooling unit shown in *Figs 2* also allows low-temperature mixes to be made.

Testing Procedure

Most of the data quoted in this Paper were obtained from the following observations on each plant, made so far as possible during normal commercial work :—

(*a*) Grading determination of two samples per day for 5 days taken from aggregate stockpiles or from crusher output.

(*b*) Samples as for (*a*) from hot-stone or dry-stone storage bins.

(*c*) Analysis of samples of mixed materials, (i) from ten consecutive mixes; and (ii) from two mixes per day for 5 days.

(*d*) Analysis of samples of mixed materials from the ends and middle of the mixer after several different mixing times.

(*e*) The output of driers and heaters, which were recorded together with the oil used and the temperature of the heated stone.

(*f*) The recorded temperatures of about fifty consecutive batches of mixed materials.

(*g*) A record of the time at which each mix was completed throughout a busy day's work.

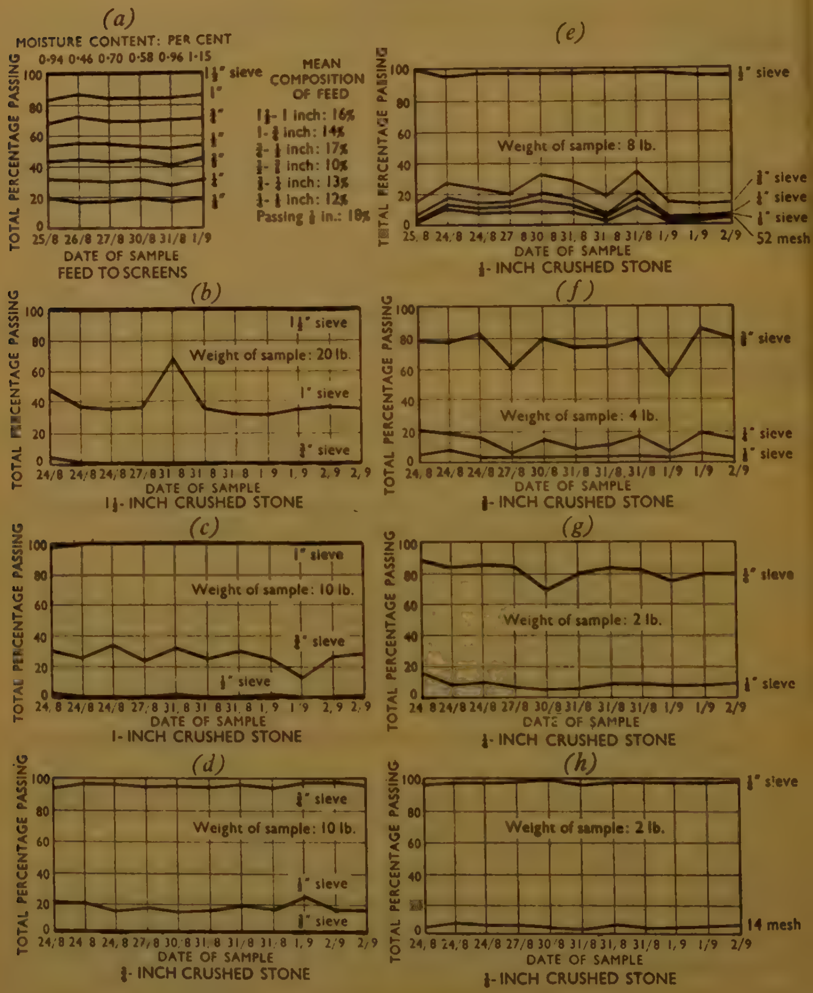
(*h*) Observations made on the utilization of labour, on those plants which were working as independent units.

It should be noted that sampling and analysis tests have shown that when the same batch of $\frac{3}{4}$-inch carpet material was sampled ten times by the method used in the present work, the variation in binder contents was found to range from 3·9 per cent to 4·3 per cent (that is, $\pm$ 0·2 per cent), and the maximum apparent variation in the amount of aggregate passing $\frac{3}{8}$-inch mesh was from 29 per cent to 39 per cent (that is, $\pm$ 5·0 per cent). These figures should be borne in mind while reading this Paper.

Aggregate Feed

Coated-macadam plants usually receive their aggregate direct from the crusher of the quarry at which they are situated. Gradings of samples from the crusher output of plant No. (2) are shown in *Fig. 3 (a)*. It will

Figs 3



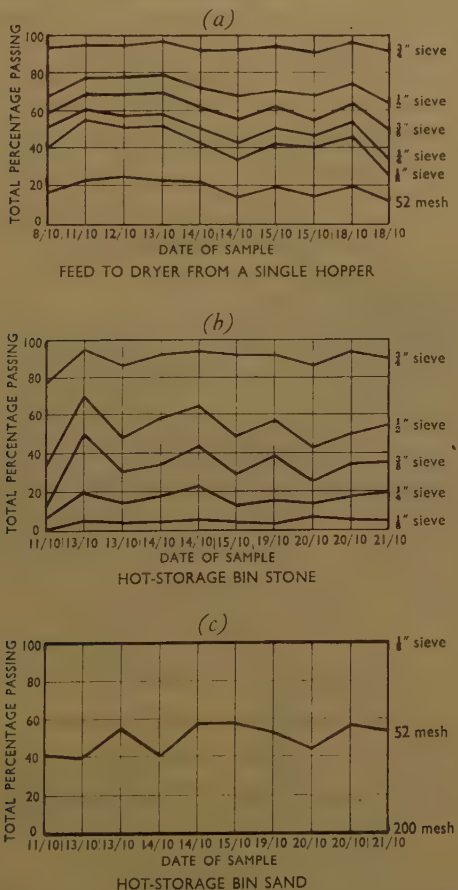
Gradings of Samples taken over a Period of a Week from the Feed from the Crushers and from the Dry-Stone-Storage Bins on a Modern Coated-Macadam Plant (Plant No. (2)).

The screen arrangement on this plant is shown in *Fig. 6 (a)*.

be seen that no size varied by more than $\pm 2\frac{1}{2}$ per cent during a period of one week. It can therefore be concluded that under normal circumstances variations in this form of aggregate-supply are unlikely to have an adverse effect on the final grading of the mixed materials.

Asphalt plants usually receive their aggregate by road or rail, graded

Figs 4



VARIATION OVER A PERIOD OF A WEEK IN THE GRADING OF THE FEED AND OF THE AGGREGATE IN THE HOT-STORAGE BINS OF AN ASPHALT PLANT (PLANT NO. (7))

Screen arrangement on plant is shown in *Fig. 6 (b)*

into certain specified sizes. The final grading will depend to some extent upon the grading of the material supplied, which is sometimes very variable. Since asphalt plants have no large storage capacity, it is necessary to feed continuously approximately the grading required in the asphalt. When a

single-feed hopper was used, the gradings shown in *Figs 4* were obtained. Lack of balance in this feed caused periodic overflowing of the sand hot-storage bin with consequent delay to the plant. (See *Fig. 15* (*c*), p. 166.)

Plant No. (4) had two feed hoppers, one for sand and the other for stone, but since three sizes of sand and several sizes of stone were being fed the combined feed was still very variable.

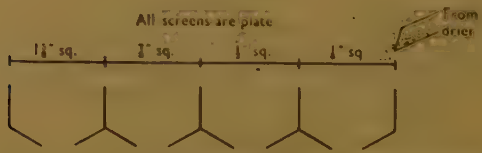
There seems to be little doubt that, for satisfactory feeding of an asphalt plant, there should be a separate feed hopper with controlled outlet for each size of stone fed to the plant. Such an arrangement, shown in *Fig. 5*, has been seen recently by the Author in the United States and it appeared to be working very well. In that case, four hoppers filled by a large bulldozer were feeding aggregate simultaneously to four asphalt plants, which were producing a total of 3,500 tons of asphalt per day.

Screening and Storing

In coated-macadam plants it is usual to screen the feed into about ten sizes and store each size in large bins, each of which might hold up to several

Figs 6

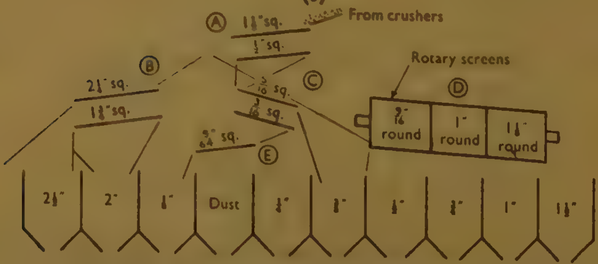
(*a*)



Each plate is 3ft square, without inclination, and vibrates at 480 osc. per min.

ASPHALT PLANT NO.(4)

(*b*)



(A) 3½′ × 8½′ at 15-degree inclination, 750 r.p.m.
(B) 3½′ × 6½′ at 17-degree inclination, 1,000 r.p.m.
(C) 4′ × 6½′ at 25-degree inclination, 300 r.p.m.
(D) 4′-dia. × 28′ at 4-degree inclination, 112 r.p.m.
(E) 4′ × [illegible] at 30-degree inclination, 3,000 r.p.m.

COATED MACADAM NO. (2)

Examples of Screening Systems on a Portable Asphalt Plant (Plant No. (4)) and on a Large Coated-Macadam Plant (Plant No. (2))

Fig. 5

Elevators and Controlled-Output Hoppers blending Four Sizes of Aggregate to be fed simultaneously to Four Asphalt Plants having a Combined Output of 3,500 Long Tons per Day. (Construction Work on New Jersey Turnpike, 1951)

Fig. 9

Electrical Metering Units attached to Each of the Dry-Stone Storage Bins of Plant No. (3)

Fig. 10

Twin-Shaft Paddle Mixer

hundred tons of aggregate. These bins are drawn on, both for the mixing plants and for the sale of dry stone.

The arrangement of the screens on a plant (No. (2)) is shown in *Fig. 6* (*b*) and the gradings produced over a period of one week in seven of the bins are shown in *Figs 3*. It will be seen that the grading is relatively consistent in all the bins except the one containing ½-inch stone, where the overloading of the ½-inch screen has diverted smaller sizes, including dust, into this bin.

The screens on asphalt plants are often simpler than those on coated-macadam plants, since they frequently consist of a single vibrating trough as shown for plant No. (4) in *Fig. 6* (*a*). All the feed passes over the smaller mesh and then on to the larger meshes in succession. For a rate of feed of about 19 tons per hour the proportion of sand carried over into the ½-inch bin ranged from 11 per cent to 26 per cent, suggesting that in this arrangement the finer screens were continuously overloaded.

A glance at the final compositions shown in *Figs 7* would show that in the present case little or no advantage was gained by sieving the aggregate into four sizes (plant No. (4)), as compared with sieving it into only two (plant No. (7)).

Proportioning the Aggregate

There are two main classes of proportioning devices—those which weigh the aggregate and those which measure it by volume.

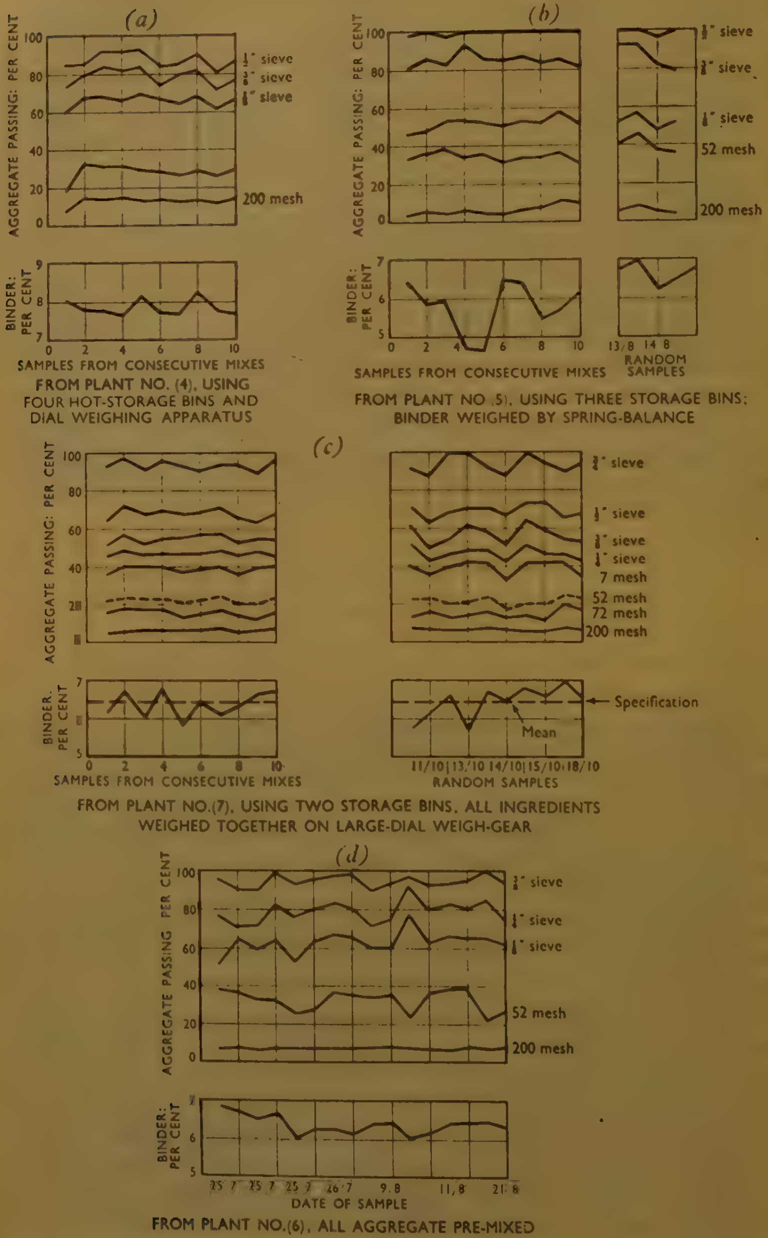
Weigh-batchers.—The weighing devices examined were of two forms: those suspended from a steelyard and those having a dial-and-pointer type of weigh-gear.

Of the two methods of weighing, the dial-type was the better because the operator could see how rapidly the aggregate was flowing into the weigh-hopper throughout the whole batching process. In no case was the operator able directly to check the accuracy of the dial-type weighing machines; some idea of their accuracy can be obtained from the amount of material passing No. 7 mesh in *Figs 7* (*c*), where the grading of sand and stone were very consistent.

With the steelyards there was a tendency to overshoot the required weight. On one plant a series of steelyards could be brought into action successively, one to weigh-out each size of aggregate. It was found during tests on this plant that the required weight of aggregate was sometimes in the hopper before any of the last size had been weighed in.

Volume-proportioning devices.—Three types of volume-proportioning devices were used on the plants included in the survey. The first and oldest consisted of a hopper which the mixer-man filled by eye to a marked level from two storage bins. The second was an electrical device which measured out complete batches from six or eight bins automatically. The

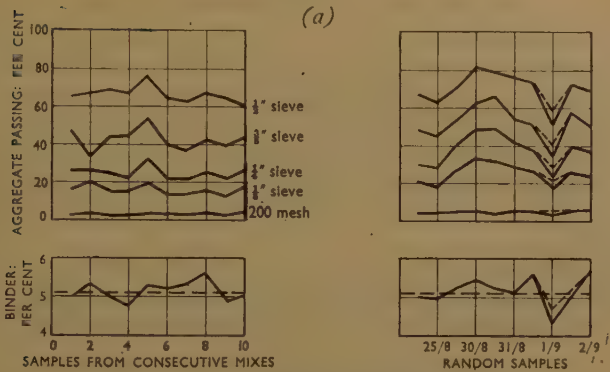
Figs 7



VARIATIONS IN THE COMPOSITION OF ASPHALT MADE IN FOUR OF THE PLANTS EXAMINED

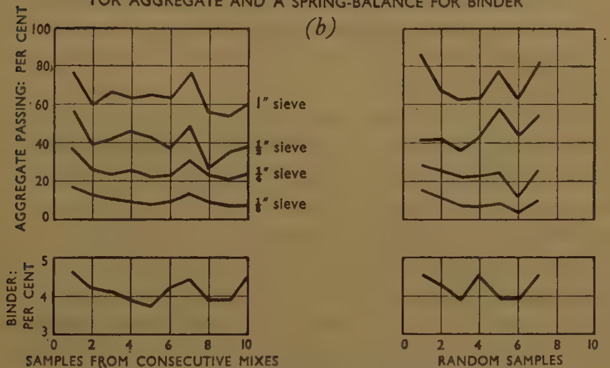
Figs 8

(a)



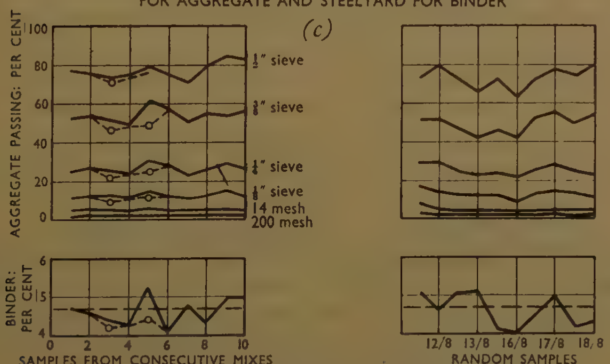
VARIATION IN THE COMPOSITION OF A CARPET MATERIAL
MADE BY PLANT NO.(2), WITH STEELYARD-WEIGHING
FOR AGGREGATE AND A SPRING-BALANCE FOR BINDER

(b)



VARIATION IN THE COMPOSITION OF 1½-INCH COATED MACADAM
MADE BY PLANT NO.(1), USING VOLUME-MEASUREMENT
FOR AGGREGATE AND STEELYARD FOR BINDER

(c)



VARIATION IN THE COMPOSITION OF A CARPET MATERIAL
MADE BY PLANT NO.(3), WITH ELECTRIC AGGREGATE-GRADING
AND A STEELYARD FOR PROPORTIONING BINDER

Variations in the Composition of Coated Macadam made in Three of the Plants examined.

third was a gate and conveyor, which controlled the aggregate flow on the continuous mixing plant.

No direct tests were made on the proportioning produced by either the simple hopper or the device on the continuous mixing plant, but the gradings of the mixed materials produced by these plants were determined and are shown in *Figs 8* (*b*) and *Figs 7* (*d*) respectively. In both cases the overall proportion of aggregate to binder appears to have been consistent, but the grading could not be controlled by either of these devices and it was very variable.

An electric grading device is shown in *Fig. 9* (facing p. 151). The compositions produced by this electrical device, and shown in *Figs 8*, suggest that it is accurate enough to meet modern specifications.

Conclusions Relating to the Handling and Proportioning of the Aggregate

1. Variation in the grading of the feed on the asphalt plants was greater than for the coated-macadam plants; it arose from variations in the aggregate as delivered and from the difficulty in combining the sizes in a single-feed or even a double-feed hopper. A controlled-output feed-hopper should be provided for each size of aggregate fed to the mixer.

2. The screening systems on the coated-macadam plants appeared to be satisfactory with the exception that on one plant a single screen was overloaded.

3. The screens on the asphalt plants were all working near their maximum capacity and this resulted in occasional carry-over of a substantial proportion of undersize material.

4. The large number of bins used on coated-macadam plants appears to meet the present very variable demand for the different grades of material.

5. When two hot-aggregate storage bins were used on an asphalt plant the gradings were just as consistent as those obtained when four bins were used.

6. The electrical grading device on one of the coated-macadam plants gave a consistent grading and it was easy to change from one material to another.

7. Batching by large dial-type weighing machines gave satisfactory results. The steelyard type was not nearly so satisfactory and should not be fitted to new plants.

8. There is an urgent need for the development of a method of checking proportioning devices directly and as a matter of routine (for example, by the use of standard weights if these were not too cumbersome and could be handled mechanically). This is particularly important when automatic methods of batching are used.

Proportioning the Filler

On all the modern plants examined, some form of filler or fine aggregate was added to the mix. Observation on the mixer platform showed that the amount of filler varied between one-half and twice the specified quantity and this effect can be seen in some of the analysis results.

Irregularity of the flow when the bin-door was opened appeared to be the chief difficulty in the batch-plants. Sometimes a complete stoppage would be followed by a sudden flow while the door was open and as much as twice the required amount of filler might fall into the weigh-hopper. The accuracy of graduated gate and conveyor used on the American continuous plant did not differ significantly from that of the batch methods ; the reason for the irregularities in this case was not determined.

The development of an effective method of proportioning filler is urgently required.

Proportioning the Binder

In a recent Paper, Dr A. R. Lee [1] has shown that of all the factors affecting the behaviour of bituminous road surfacings the amount of binder has the most important effect upon the properties of the finished surfacing. Large differences occur in the lives of surfacings of any particular type ; such differences arise chiefly from variations in binder content.

Four types of apparatus were used to proportion the binder on the plants examined—a steelyard weighing device, a simple spring balance, two lever weighing machines with dial-scales 14 inches and 18 inches in diameter, and a volumetric pump on the continuous plant. Except in the lower part of the range, none of these devices could have their accuracy checked by any simple direct means such as the use of standard weights. Analysis results are of only limited value for checking accuracy of proportioning because they also depend upon the accuracy of proportioning of aggregate and filler and on variations in the sampling and analysing processes.

The continuous mixer with volume-proportioning of binder gave considerable trouble before the correct binder-content was obtained, but with a team of research workers on the site the results shown in *Figs 7* (*d*) were obtained.

Conclusions Relating to the Proportioning of Binder

Despite the difficulty of making direct measurements on the proportioning devices, the compositions shown in *Figs 7* and *8* together with a few tests made with small standard weights lead to the following conclusions :—

1. Small (domestic-type) spring-balances are not suitable for proportioning binder on mixing plants.

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[1] A. R. Lee, " Full-Scale Experiments in Road Research, with Special Reference to Thin Carpets." Road Paper No. 31, Instn Civ. Engrs, 1950.

2. Steelyard weighing devices tend to give variable results because zero adjustments are not made sufficiently often and because of the tendency occasionally to over-shoot the balance point.
3. Dial-type weighing apparatus fitted with a large clear dial gives accurate results. This accuracy must always be dependent upon the skill of the mixer-man and the correct functioning of the relatively complex system of levers working under conditions of test and vibration.
4. Taking into account all the errors, including those of sampling and analysis, the plants investigated are capable of giving binder contents within the range of $\pm 0{\cdot}5$ per cent of the specification, and in four of the cases observed the mean of twenty analyses was within 0·1 per cent of the specification.
5. Continuous mixers can give as constant a binder content as batch plants after they have been adjusted skilfully, but they are most suitable where one specification is to be mixed continuously.

It is desirable, if more consistently high standards of construction are to be reached, that a simpler and more accurate method of batching binder should be developed. Russell[1] has shown that a simple loaded piston and cylinder can be arranged to give accurate volume-batching of binder, and there is an American "fluidometer" which will meter-out batches of binder automatically. There seems little doubt that such automatic methods will be favoured in the future.

Mixers

The mixers examined were of the twin-shaft paddle-type, an example of which is shown in *Fig. 10* (facing p. 151). The factors considered were the effect on uniformity and speed of mixing of the arrangement of the blades, the speed of rotation of the shafts, the method of tipping in the aggregate and binder, and the shape and position of the outlet door.

Figs 11 show the compositions of samples of mixed materials taken from the ends and the centres of three of the mixers. In the mixer of plant No. (5) the blades were not arranged to impart movement parallel to the shafts and, because the coarse and fine aggregates were put in at opposite ends and the outlet door stretched the whole length of the mixer, the segregation shown in *Figs 11* persisted throughout the process. The mixer of plant No. (1) was similar to that of plant No. (5), but the materials were tipped-in evenly along the axis of the mixer. The mixer on plant No. (2) had the blades arranged so as to impel the material towards the centre and also had a central outlet door.

In recent years, higher speeds of rotation of the shafts (29–39 revolu-

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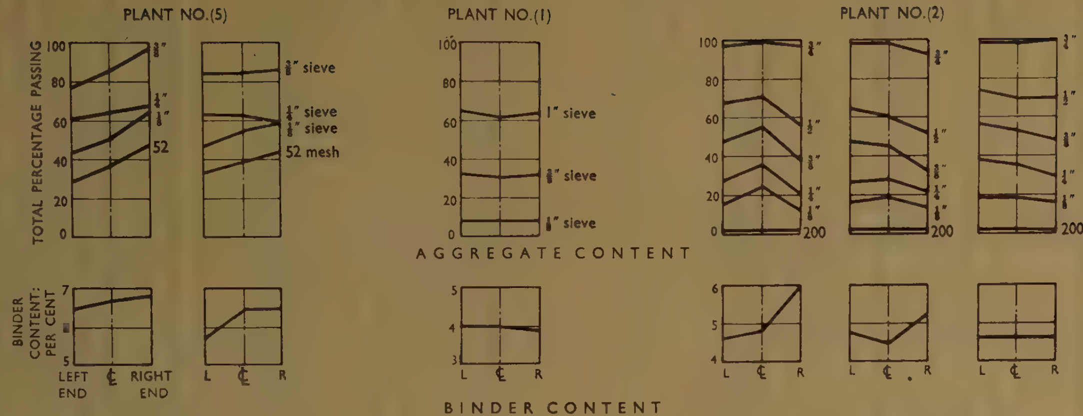
[1] W. L. Russell, "Automatic binder batcher for an asphalt plant." *Rds & Rd Constr.*, vol. 28, p. 178 (June 1950).

Figs 11



ROLLED ASPHALT

After 1 min. mixing *After 2 min. mixing*

(a) Mixer having uneven feeding and no longitudinal mixing

1½-INCH TARMAC

After 3 min. mixing

(b) Mixer having well-distributed feeding but no longitudinal mixing

¾-INCH CARPET MATERIAL

After 20 sec. mixing *After 40 sec. mixing* *After 80 sec. mixing*

(c) Mixer having appreciable mixing from ends to middle

RESULTS OF ANALYSIS OF SAMPLES TAKEN FROM THE ENDS AND THE MIDDLE OF THE THREE MIXERS AFTER DIFFERENT MIXING TIMES.

(Each diagram refers to a different batch.)

tions per minute) have been used in Great Britain. It is quite common for speeds of 70 revolutions per minute to be used in the United States of America. The use of the high mixing speeds is associated with correspondingly shorter mixing times and therefore higher outputs.

Conclusions Relating to Mixers

1. If the mixer is correctly designed and the feed arrangements are satisfactory, twin-shaft paddle-type mixers can give a uniform mixture with regard to both grading and binder content.

2. Best mixing is obtained when the faces of the paddle blades are so arranged that they produce longitudinal mixing of the material in addition to rotation. If there is no such longitudinal mixing it is essential that all the materials are spread evenly along the mixer, and the outlet door must also stretch the full length of the mixer.

3. Some form of time-lock or alarm mechanism should be fixed on mixing plants to ensure that each batch is mixed for a sufficient time to become uniform.

4. It is suggested that a method of test similar to that used for mixers in the present work might be adopted to measure the uniformity of mixed materials.

Drying and Heating the Aggregate, and Temperature Control

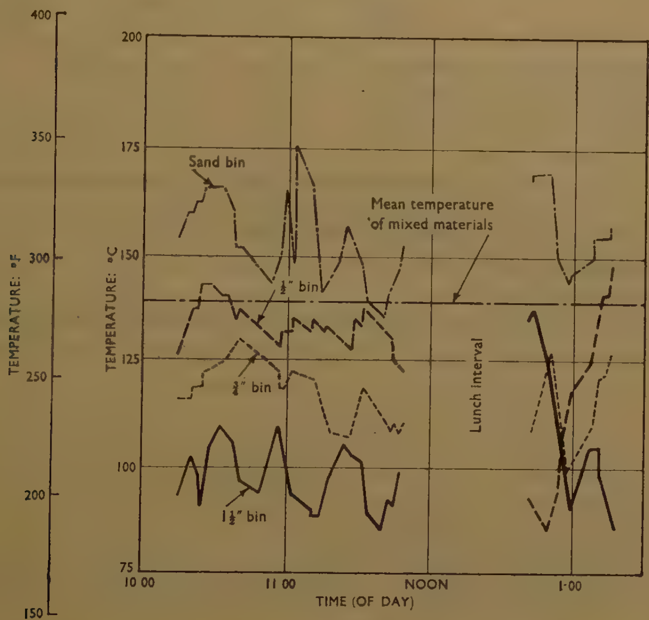
Tests were made on three continuous driers, three batch heaters, and a cooling unit. Before discussing the work it is necessary to point out a difficulty which arises with all attempts to measure the temperature of aggregate on a mixing plant. The difference in the sizes of stone or sand passing through the heater for initial heating results in temperature differences (as shown in *Fig. 12*) which persist until after the particles have been mixed together for several minutes. The existence of this temperature range within the aggregate means that temperature measurements should be made preferably on the mixed material or on aggregate which has been allowed to come to a uniform temperature.

Continuous driers.—In testing each drier, the output of mixed materials, the temperature and moisture content of the aggregate, and the amount of fuel-oil used were all measured. Each test lasted for at least one hour, the period being selected from a long run of normal commercial work. The results of the tests are included in Table 1 (p. 160).

While in America recently, the Author saw driers (not very much larger than those examined in the present work) which were drying and heating aggregate to a temperature of 275° F. (135° C.) at a rate of 800 tons per day, using 1·83 Imperial gallons of oil per ton as compared with an output of 200 tons per day with a fuel consumption of 2·2 gallons in Great Britain. The American plant was working in very hot weather so that the thermal efficiencies of British and American driers are probably not very different,

but the higher outputs obtained by the latter are obtained by burning a correspondingly larger amount of fuel. The use of burners with so greatly increased a capacity may require modifications to be made in the design of the combustion chamber. The drier frequently provides the bottle-neck on British plant (see Table 2, p. 167), so that a substantial increase in output is desirable.

Fig. 12



TEMPERATURES OF THE DIFFERENT AGGREGATE SIZES AFTER THEY HAVE BEEN SEPARATED INTO THE HOT-STORAGE BINS OF AN ASPHALT PLANT (PLANT NO. (4))

Batch heaters.—Each of the three batch heaters tested was 6 feet in diameter by 4 feet in length; each had five internal lift-plates and rotated at 12 revolutions per minute.

Results of tests on the batch heaters are shown in Table 1.

The explanation suggested for the variation in the amounts of oil used in these almost identical heaters was that in plant No. (2) the flame was developed inside the drum, so that a proportion of the burning gases passed immediately to exhaust, whereas in the heaters of plant No. (3) the flames were developed in a combustion chamber before passing into the drum. The difference in consumption between the heaters in plant No. (3) arose from the fact that in one the flame was kept low and the aggregate remained in the heater as long as the time-cycle at the mixer

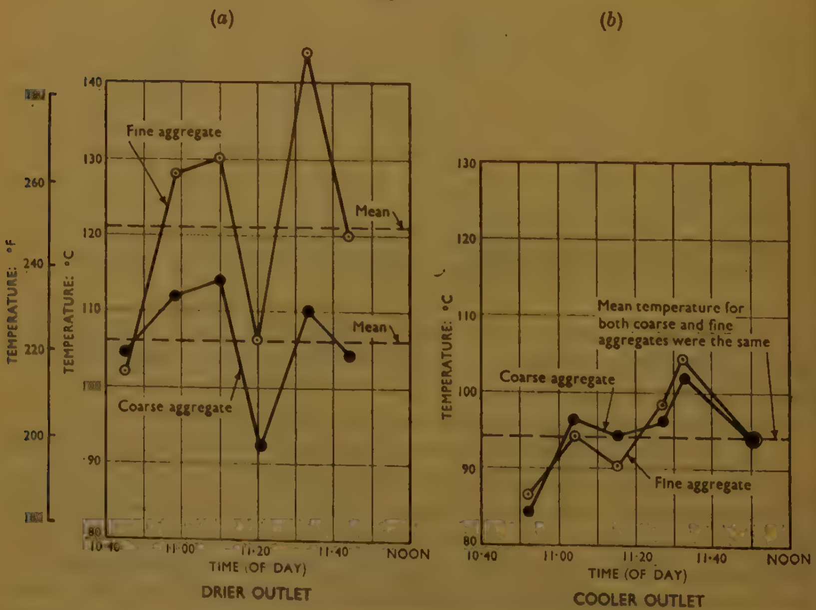
TABLE 1.—RESULTS OF TESTS ON DRIERS AND HEATERS

Plant ref. No.	Mixing temp.: °C.	Mixed material: tons per hour	Oil used: gallons per hour	Oil used: gallons per ton of product	Remarks
Batch heaters					
2	50–60	32·0	12·5	0·39	
3	,,	19·0	4·25	0·22	Duplicate heaters on same plant
3	,,	19·0	7·75	0·41	
Continuous driers					
7	132	20·5	32·0	1·5	
4	193	19·0	43·5	2·3	
5	170	16·6	43·0	2·2	

allowed, whereas in the other the flame was kept high and each batch remained in the heater for only a short time, the heat going to waste for the remainder of the time-cycle.

The cooling unit.—The cooling unit examined is shown diagrammatically in *Figs 2*. It consisted of a cylindrical steel casing, about 5 feet in diameter

Figs 13



THE TEMPERATURE OF THE COARSE AND FINE FRACTIONS OF THE AGGREGATE BEFORE AND AFTER PASSING THROUGH THE COOLING UNIT (PLANT NO. (7))

and 12 feet high ; it contained internal baffle plates over which the aggregate flowed by gravity. Cold air was blown into the cooler at different levels and escaped at the top.

In the test, which was carried out at the nominal capacity of this unit (10 tons per hour), buckets of aggregate were taken at the same moment from the drier outlet-chute and from the cooler outlet-chute. These aggregate samples were put on a sieve and divided immediately into sand and stone portions and the temperature of each was determined. The results of this work are shown in *Figs 13.*

Substantial cooling cannot be expected in the 5 or 6 minutes during which the aggregate might remain in a cooler of the type tested, because the flow of heat from any object depends upon the difference between its surface temperature and that of the air immediately in contact with it. In a heater, the hot gases would be several hundreds of degrees above the stone temperature so that the stone could be heated quickly, but the temperature difference in a cooler would seldom exceed 50° C. (90° F.).

For greater temperature reduction the cooler would have to be very much bigger, so that it could, when necessary, hold a larger quantity of aggregate, and an individual stone would take longer to pass through.

Temperature control.—In order to assess the effectiveness of the temperature control on the plants examined, the temperature was recorded of every mix made in a period of several hours during a busy day's work. The results of these observations are shown in *Figs 14.*

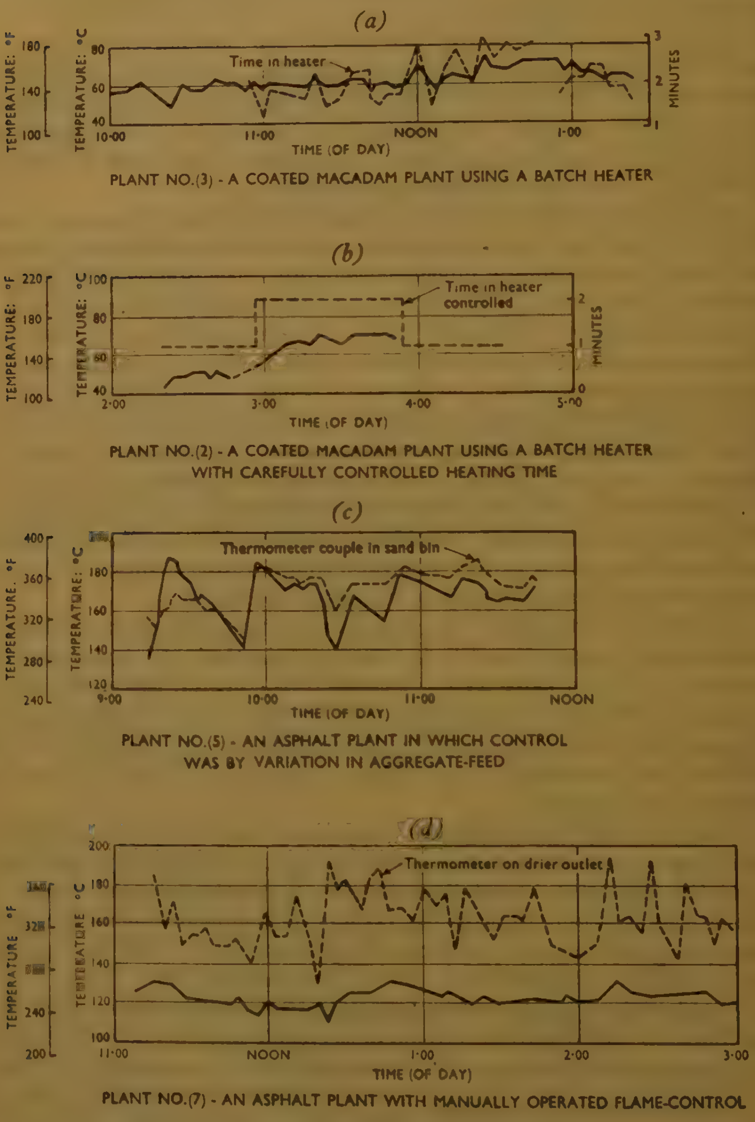
For batch heaters it will be seen that the temperature of the batch is approximately proportional to the heating time, but with some slight lag caused by the heat capacity of the shell of the heater, the elevators, and the mixer. This general proportionality had been realized by the operators of plant No. (2) and they controlled the heating time by means of an alarm bell and thus exercised control within a range of 5½° C. (10° F.) once the lag had been overcome.

On two of the asphalt plants temperature control on the continuous driers was by manual operation of the oil-supply to the flame ; on the third plant the flame was kept full-on and the aggregate feed was varied. It will be seen that only in plant No. (7) was the degree of control other than very poor, the range of variation being about 50° C. (90° F.) in plants No. (4) and No. (5) and 20° C. (36° F.) in plant No. (7).

On plant No. (7), the metal-clad thermometer in the drier outlet-chute was partially exposed to the stream of aggregate and this made for quicker and more definite response. The man responsible for control was seldom away from the oil valve because his thermometer reading was constantly changing. The maximum temperature-variation over the best hour of work on this plant was 8° C. (14° F.).

The results suggest that some form of automatic temperature control should be possible, and that by such means wide cyclical variation in temperature at the drier outlet might be completely smoothed out by the

Figs 14



Variation in Temperature of the Mixed Materials produced in Five of the Plants included in the Survey

time the materials have been mixed. To allow such a controller to work effectively it would be necessary to ensure a constant rate of feed of the fine and coarse aggregate.

Conclusions Relating to Driers and Temperature Control

1. Batch heaters form a convenient method of heating aggregate for coated macadam, particularly when many different grades of material are to be made consecutively. Some attention is needed to obtain the optimum thermal efficiency.

2. The output of British continuous driers is much less than that of corresponding American driers, mainly because the British types use smaller oil burners. Since continuous driers on British plant are frequently the bottleneck limiting production, it is suggested that their maximum output should be substantially increased.

3. The cooling unit on the dual-purpose plant removed temperature differences between the different-sized aggregate particles and reduced the average temperature by about 20° on the Centigrade scale (36° on the Fahrenheit scale).

4. The application of agreed methods and conditions of test for driers and heaters would help to stimulate improvement in output and efficiency both in the operation of existing plant and in new design. At the time of writing, consideration is being given to this problem by the British Standards Institution.

5. Temperature control on batch heaters gave little trouble because the temperature of each batch was approximately proportional to the time the batch spent in the heater. A maximum range of variation in commercial work of $\pm$ 5° C. ($\pm$ 9° F.) should be practical for mixes under 100° C. (212° F.).

6. Temperature control on continuous rotary driers was considerably less satisfactory than for batch heaters. The best one gave a variation of $\pm$ 10° C. ($\pm$ 18° F.) and the worst $\pm$ 25° C. ($\pm$ 45° F.). The results showed that it was better to control the oil burner than to control the rate of feed of aggregate. Manual control could only be effective with constant attention. Research work is now in progress on the development of an effective method for automatic control.

7. When testing the effectiveness of temperature control, measurements should be made on at least twenty successive batches of mixed materials using a mercury-in-glass or metal thermometer having a low heat-capacity.

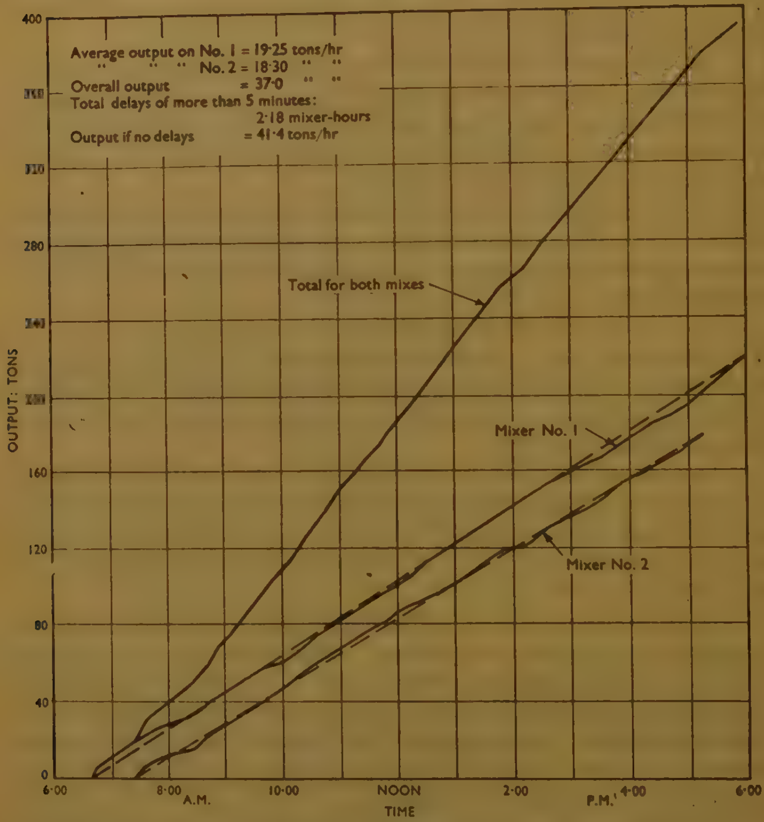
Output Measurements

The overall output of each plant was measured by recording the time at which each mix was completed throughout several hours' work, sometimes for a full day. The results of some of these observations are

illustrated in *Figs 15* and results obtained from five of the plants are summarized in Table 2.

The outputs of the asphalt plants were approximately 20 tons per hour and of the coated-macadam plants 30 to 40 tons per hour. The need for increasing the potential output of all mixing plants is realized when these

Fig. 15 (a)



MEASUREMENTS OF OUTPUT OF MIXED MATERIALS FROM PLANT NO. (3) DURING A BUSY DAY'S WORK

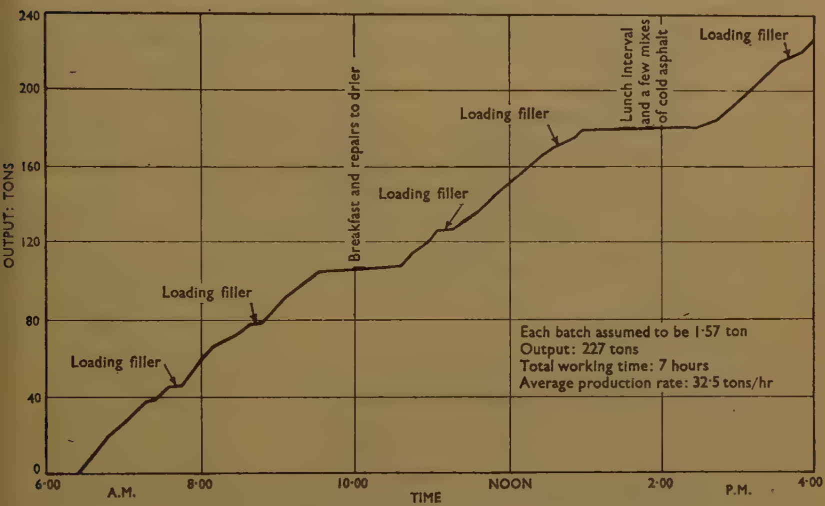
outputs are considered in relation to the capacity of a mechanical finisher, which, travelling at its lowest speed, will easily lay an average of 30 tons per hour of rolled asphalt and substantially greater tonnages of coated macadam.

In the case of the two coated-macadam plants included in the survey, the output of the plant was limited by the output of the quarries in which they were situated. A substantial increase in output would require changes

in quarry output and the discussion of such matters is beyond the scope of this Paper.

The rolled-asphalt plants provided a simpler case because they drew their supplies from stockpiles and each plant was used to supply one finisher. On such plants there would be no insuperable difficulty in organizing the supply of materials to meet a greater output. It is suggested therefore that it should be possible to increase the output from the

Fig. 15 (*b*)



Measurements of Output of Mixed Materials from Plant No. (2) during a Busy Day's Work

present 20 tons per hour to about 30 tons per hour or more without rendering the plants immobile or making an undue increase in their size or complication.

To find out whether or not such an increased output is feasible, the throughputs of the constituent units of the different plants were examined. On each plant surveyed, although it was possible to observe which constituent unit was limiting the output, it was more difficult to determine the possible throughput of those units which had the higher capacities, since they could not be tested under their full load. A careful estimate, based on direct observations of time-cycles, was made in each of these cases; for example, the maximum output of a mixer on an asphalt plant would be judged by the average time taken to complete a number of individual mixes.

Measurements and estimates of the throughput of the constituent units of five of the plants surveyed are included in Table 2.

Conclusions Relating to Output

1. The outputs of all the plants tested were rather small compared with the potential laying of the modern road-finisher. An increase in output of at least 50 per cent is therefore desirable, particularly from the plants which supply wholly to one site.

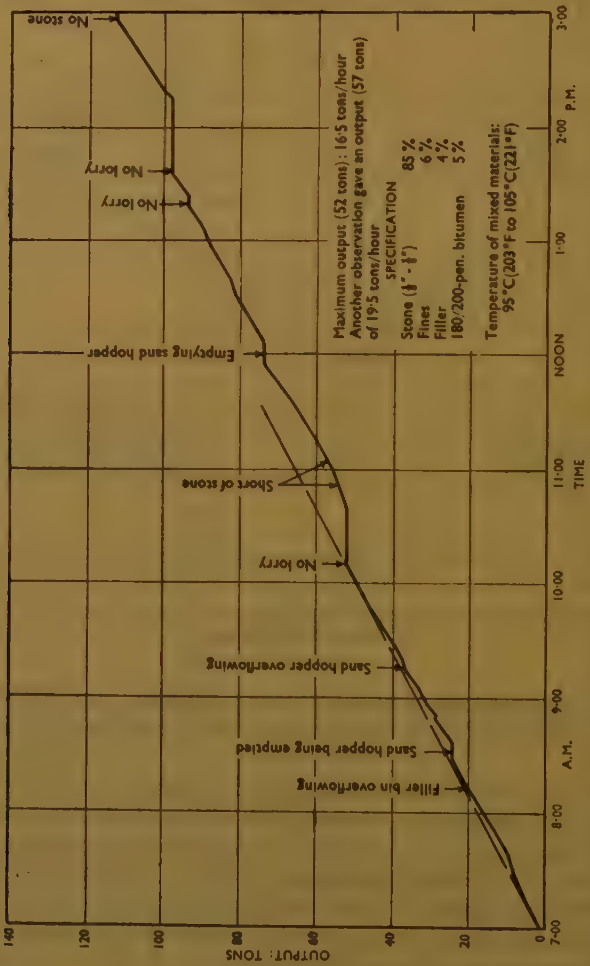


Fig. 15 (c)

MEASUREMENTS OF OUTPUT OF MIXED MATERIALS FROM PLANT NO. (7) DURING A BUSY DAY'S WORK

2. The constituent units of the coated-macadam plants were fairly well balanced and to obtain a large increase in output it would first be necessary to increase the output of the quarries supplying them. When making carpet material, the batch heater was usually the bottleneck, but

TABLE 2.—MEASURED OR ESTIMATED MAXIMUM THROUGHPUTS OF FIVE OF THE PLANTS SURVEYED AND OF THEIR CONSTITUENT UNITS

Unit	Maximum throughput: tons per hour						
	Plant (4) (asphalt)		Plant (5) (asphalt)	Plant (7) (dual purpose)	Plant (2) (coated macadam)		Plant (3) (coated macadam)
Type of output	Asphalt base	Asphalt surfacing	Asphalt surfacing	Carpet	¾″	1½″	Various grades
Measured output	19·9	20	19·6	17·8	32·5		37·0
Possible output (no avoidable delays)	20	20	20	20·5	28	36	41·4
Feed hopper(s)	20		60	Very large	—		—
Continuous drier	more than 20		20	20	—		—
Elevators	50		50	35	75		—
Batch heaters	—		—	—	38* (2 min. mix) 63 (1 min. mix)		47
Batching	—		—	—	79		53
Mixing	30		30	35	47		47 (two mixers)
Screens	Approx. 20 (single deck vib.)		20 (rotary)	Approx. 20 (rotary)	—		—

* Mixing carpet material.

when low temperature mixes, such as coarse macadam, were being made, the mixer set the pace.

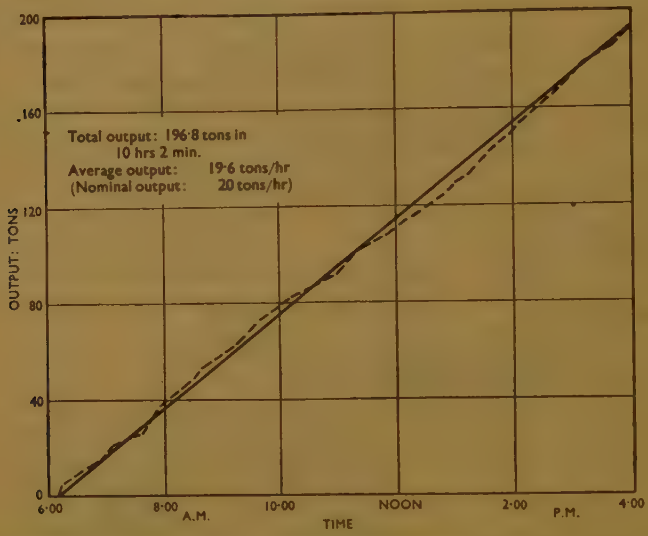
3. The chief restrictions on the output of asphalt plants were caused by the dryers and the screens; most of the other units could have dealt with a 50-per-cent increase in output with little or no modification.

4. Causes of delay on asphalt plants were chiefly lack of transport for the mixed materials; in one case, unbalanced feed made it necessary to empty one of the hot-storage bins from time to time.

5. The size of the labour forces employed was not related to the output of the plant and if the output of a plant was increased by even 100 per cent it is unlikely that more labour would be required.

6. The handling of filler in bags appeared to be a waste of manpower; consideration should be given to bulk handling of this material.

Fig. 15 (d)



MEASUREMENT OF OUTPUT OF MIXED MATERIALS FROM PLANT NO. (5) DURING A BUSY DAY'S WORK

GENERAL CONCLUSIONS

Measuring and proportioning devices on the modern plants examined were relatively satisfactory but it was noticed that operators had no ready means of satisfying themselves as to the continuous accuracy of their own plant. They depended for information mainly upon the results of analysis of samples of mixed materials, and such results do not immediately show the source of a fault. Similarly, plant manufacturers and purchasers have no generally accepted method of assessing the efficiency of the various constituent units of mixing plant.

It would seem to be an urgent requirement that all weighing and measuring devices on mixing plant be provided with a simple and direct means for checking their accuracy. Also a series of testing procedures should be drawn up and agreed upon by both manufacturers and users, in order to enable measurements to be made of the effectiveness of such units as aggregate-feeding devices, driers, temperature-control methods, screens, and mixers. Information obtained in this way would provide a sound basis for future improvements in design.

The British Standards Institution are already considering tests for

driers and some of the other tests should not be difficult to evolve, but those dealing with screens will almost certainly require basic research before a satisfactory test can be specified.

ACKNOWLEDGEMENTS

The work described in this Paper was carried out as part of the programme of the Road Research Board of the Department of Scientific and Industrial Research. The Paper is published by permission of the Director of Road Research.

The Author's thanks are due to the firms and personnel who operated the plants for their help with the investigation.

The Paper is accompanied by three photographs and fifteen sheets of drawings, from which the half-tone page plates and the Figures in the text have been prepared.

Paper No. 5884

"Aerodrome Runway Construction in Nigeria"

by

Alexander McDonald, B.Sc., M.I.C.E.

(Ordered by the Council to be published with written discussion) †

SYNOPSIS

The development of trans-African air transport, especially that between Europe and Southern Africa, has brought heavier and faster machines on the scene requiring stronger subgrades and longer runways than existed at Nigeria's two main airports, Kano and Lagos.

Runways, however, are expensive to construct, and the Nigerian economy could not and cannot afford the concrete runways which are built as a matter of course in more highly developed countries. Low-cost runways making the maximum use of local materials had therefore to be designed, and this Paper describes the design and construction procedure, together with costs. The testing of the soils and densities-in-situ obtained should be of special interest to those faced with similar problems. Details have been given of the equipment used and observations made on maintenance difficulties.

INTRODUCTION

THE increasing weights of aircraft have necessitated major extensions and strengthening of runways at practically all international airports. In Nigeria, the construction and maintenance of airfields is the responsibility of the Public Works Department, and this Paper describes works carried out to develop the two most important airports in the country, Kano and Lagos. Kano, in Northern Nigeria, is ideally situated for the handling of trans-Sahara air traffic and is almost exactly half-way between London and Johannesburg. It is the busiest airport in West Africa and is used regularly by five international airlines. Ikeja, the airport of Lagos, is the headquarters of the West African Airways Corporation; from here all the local air services radiate, and a service to London is operated six times a week. The number of aircraft, ranging from Constellations to Doves, handled at these two airports during the year 1950-51

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† Correspondence on this Paper should be received at the Institution by the 1st June, 1953, and will be published in Part II of the Proceedings. Contributions should be limited to about 1,200 words.—Sec. I.C.E.

was as follows :—

	Kano	Lagos
International services	1,606	746
Local services	1,607	2,005
Totals	3,213	2,751
Daily average	9	7·5

In this Paper an effort is made to show how full advantage has been taken of the properties of the in-situ soils to construct low-cost runways whose bearing capacities compare favourably with those in other countries where more expensive methods of construction are used. Numerous theories have been suggested for the estimation of bearing capacities of runways, but as yet no general method of design has been evolved which by itself is of any great practical value to the civil engineer. Purely theoretical methods tend to arrive at exact answers via assumptions of conditions widely different from those obtaining in actual practice. Empirical methods, which attempt to take into account the many variable factors involved, can by their very nature only suggest the wide limits between which the best solution lies. Prevailing conditions and types of construction used in Nigeria are more akin to those of America than of Britain, and much use is made of the system evolved for the classification of soils by the Highway Research Board of Washington, D.C. However, findings based on American conditions cannot be applied blindly to Nigeria (for example, frost is fortunately non-existent), and present methods of runway design are largely based on experience of the behaviour of the numerous types of soil in earlier aerodrome and road construction, and the results of many tests carried out in the laboratory and in the field.

Existing runways in Nigeria are constructed to one of three standards :

(1) Rolled and graded natural grass strips.
(2) Compacted subgrades of natural soil with overlying base and wearing surface courses of lateritic gravels up to 12 inches in thickness.
(3) Similar to (2) and sealed with two-course bituminous surface dressing.

With these types of construction, the bearing capacity of the runway is largely governed by the strength of the subsoil and in practically all cases where failures of such runways have occurred, the prime cause has been found to be subsoil weakness. Faults which may occur in the surface courses after construction can usually be remedied by the normal maintenance organization ; failure of the subsoil is often difficult and costly to repair.

The most important single factor in ensuring that these runways retain their full supporting values under all conditions is the prevention of any large increase in the moisture content of the subsoils. This can best be achieved by adequate compaction of the subgrade and the provision

of a waterproof surface. The effectiveness of each of these methods, even when applied separately, has been illustrated in the behaviour of existing airfields. The readiness with which a soil absorbs moisture decreases as its density is increased, and it has been found that if a well-graded soil is properly compacted at optimum moisture content and allowed to dry out till its moisture content is between 4 and 8 per cent, it is unlikely to saturate, even under the worst conditions. If this were not so, the runways which are not provided with an impervious surface would become unserviceable in the wet season. In practice it has been found that this occurs only in cases where the soil is poorly graded and compaction has been inadequate. Tests carried out on airfields with well compacted soils, even in depressions where water has been standing for some weeks, show very little increase in the moisture content. This suggests that methods of design which are based on saturated-soil conditions are rather too conservative in their approach. On some of the existing airfields, thin bituminous surfaces have been placed on runways which later tests have shown to have poorly compacted subgrades. The fact that these runways have stood up so well to traffic in all seasons has demonstrated the effectiveness of these thin seal-coats in preventing rain-water from penetrating the subsoils. Under Nigerian conditions, the runways require complete re-sealing every 3 to 5 years.

At Kano, where the strength of a portion of the existing main runway depends on the low moisture content of the soil rather than high density, an interesting case of moisture gaining entry below an impervious surface occurred. A narrow cable trench had been dug across the runway, and rain fell while this was still open. The trench filled with water which later disappeared into the subsoil on each side. Some time after the completion of the work, the effect on the subsoil was evident from a surface settlement of between 1 inch and 2 inches for 10 feet on each side of the re-filled trench.

The moisture content of the subsoils may also be adversely effected by groundwater or capillary water. Underground water concentrations generally occur only in depressions in impervious strata or where water flow in a previous strata is obstructed by bands of a more impervious soil. Such conditions fortunately do not exist at Ikeja or Kano, and subsoil drains are therefore unnecessary. Capillary water can only prove troublesome if the capillary rise in the soils is great enough to attract water from the saturated soils lying below the level of the water table, into the subgrades lying within about 2 feet of the runway surface. The capillary rise in most Nigerian soils is small, generally less than two feet, and the level of the water table at these two airfields is well below the surface. Surface drainage is provided by grading the runway to a suitable camber or cross-fall and a good finish to the grading is required to ensure a uniform run-off. Weaknesses have been observed at points along the hardstrip edges of bituminous-surfaced runways, where water tends to collect and

saturate the soil, and a certain amount of lateral percolation beneath the surface has resulted in increased moisture content of the subsoil. Adequate compaction of the underlying subsoil and careful maintenance of the hardstrip edges provide the answer.

Kano Airport

Kano airport is situated on a slightly domed area about 1,550 feet above sea-level, 3 miles north-north-east of the city. It was originally built in 1935 and was extended in 1942. At present it has two runways; the main one, 2,200 yards long and 50 yards wide, has a laterite base course varying in thickness from 4 to 14½ inches, and an asphaltic wearing course which has been built up by successive surface treatments of tar, bitumen, emulsion, fine gravels, and sand. No. 2 runway, 1,800 yards long, is of similar construction but has no surface dressing.

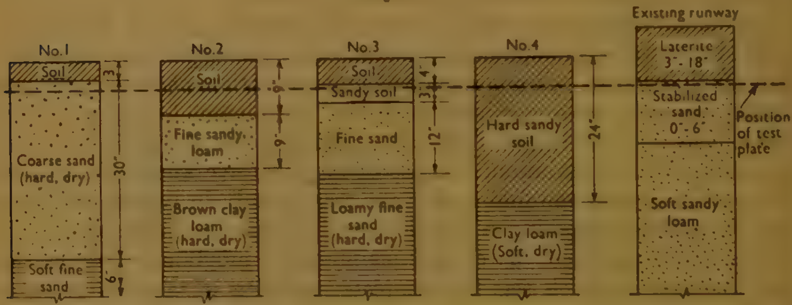
After investigations carried out by the Ministry of Civil Aviation in 1948, doubt was expressed as to the capacity of the airport for the heavier aircraft then in use. It was suggested that restrictions should be enforced during the wet season to limit the use of the airport to aircraft whose wheel loads did not exceed 80,000 lb. The Director of Public Works, as controller of Civil Aviation, because of his extensive knowledge of local soil conditions and the fact that Kano was already in regular all-season use by aircraft imposing a heavier loading than the suggested limit, advised that provided the careful maintenance programme was carried out, included in which was the complete re-sealing of the main runway, landing facilities at the airport were adequate to meet immediate requirements. However, it was realized that if Kano was to hold its place as one of the leading airports in Africa, major improvements would be required in the future, and in 1950 investigations were carried out to determine how these could best be effected. It was found that the topography of the site did not permit economical reconstruction and the extensions necessary for instrument landing by the heavier aircraft that Kano will have to cater for. The investigations were therefore directed towards providing a completely new runway of adequate length and orientation, adjacent to the airport area.

A suitable line was found about 1,000 yards to the north of the existing main runway and approximately parallel to it. The new runway will be 9,000 feet long and can be extended if necessary.

A soil survey was carried out, when inspection pits were dug on each of eight cross-sections sited on the centre-line and each outside edge of the proposed runway. The pits were dug to a depth of 18 inches below the proposed surface level or until the bottom horizon was reached, whichever was the deeper. The levels of the different horizons were related to the datum survey levels, and soil samples from each horizon were examined and tested in the Materials Laboratory in Lagos.

While the design of the new runway was being considered, the M.C.A. testing party, which had previously carried out tests at Ikeja, visited Kano to estimate the bearing capacity of the existing runways. Advantage was taken of this visit to gain more information of the subsoils of the new runway. Tests were carried out at four points along the proposed centre-line, on the natural subgrade after the removal of the topsoil. The 30-inch-diameter plate was used and one repetition of load applied at contact pressures of about 20 lb. per square inch. The results are shown in *Figs 1.* The initial cycle of loading is equivalent to light compaction of the subsoil, and it will be seen how markedly the bearing capacities are improved by this. In fact the second cycle of loading gave better results than those obtained in tests on the subsoil of the existing main runway, and showed that the bearing capacity of the subsoil in its compacted state

Figs 1



LOAD-BEARING TESTS (ON 30-INCH-DIAMETER STEEL PLATE). KANO AIRPORT

Deflexion : inch	Cycle	Load : lb. or Pressure : lb./sq.in.	Proposed new runway				Existing main runway
			No. 1	No. 2	No. 3	No. 4	Average results on subsoil
0·05	First	Load	6,500	15,800	16,000	17,000	19,000
		Pressure	9·3	22·3	22·6	24·0	26·8
	Second	Load	28,000	34,800	29,000	38,000	34,000
		Pressure	39·5	49·2	41·0	53·7	48·0
0·1	First	Load	16,000	31,000	26,000	>32,000	33,000
		Pressure	22·6	43·8	36·7	>45·2	46·6
	Second	Load	43,000	49,000	41,000	>50,000	37,000
		Pressure	60·7	69·3	58·0	>70·6	52·3

was not much lower than that of the existing runway surface. It seemed clear, therefore, that, with adequate compaction, the natural subsoil would be suitable as a subgrade for the new runway.

The importance of an exact knowledge of the nature of the soils in the different horizons and of the varying depths of these, has been well illustrated in Nigeria. In the construction of Yelwa aerodrome, during the process of running the soil from cut to fill, the horizons were reversed. The result was that the upper horizon of good granular material was covered by a lower clay horizon, and the surface as constructed was unserviceable in wet weather. It should be mentioned that the aerodrome at Yelwa was built in war-time when speed of construction was the criterion.

Typical analyses obtained from the soil survey are shown in *Figs 2* and *11* (p. 183), and from these it will be seen that the top 6 inches of soil are inferior to those below. This is confirmed by the M.C.A. bearing tests; the best results were obtained in test No. 4, where there existed 24 inches of hard sandy soil overlying clay loam, or where soil of the type found generally in the second horizon was of the greatest depth and lay directly under the testing plate. Average results of the density tests carried out over the whole area were :—

	Top 6 inches of soil	At 12 inches depth
Maximum dry density . .	110	120
Optimum moisture content .	10–12 per cent	10–12 per cent
Density in situ	90	90

It was decided that the top 6 inches of soil should be removed over the whole hardstrip area and that any filling necessary on this area should consist of soil taken from below this depth.

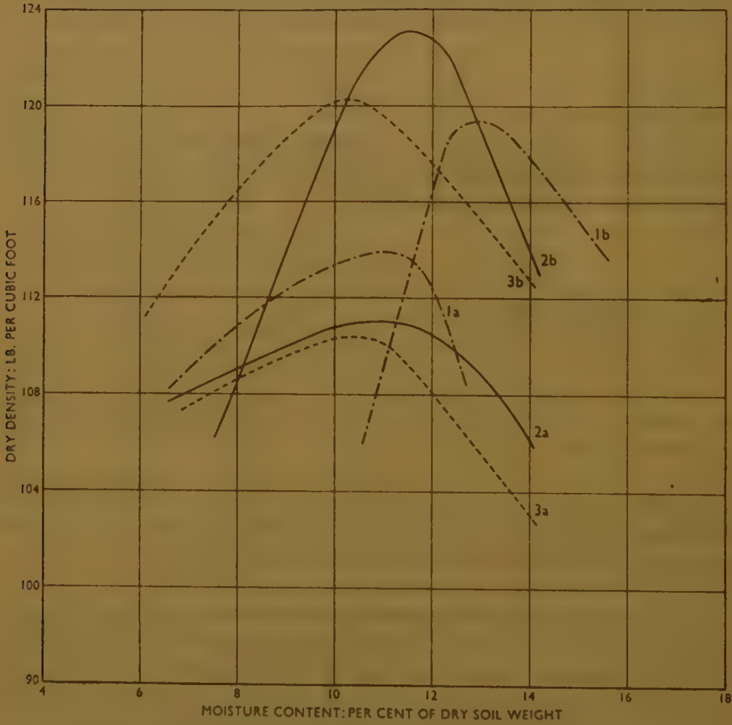
Mr R. W. Taylor, C.M.G., late Director of Public Works, Nigeria, in his Technical Paper No. 11 on aerodrome subsoils, based on extensive investigations into Nigerian airfields during 1943, wrote :—

> "Hitherto it has been assumed that a runway formed in cut is beneficial, and that there is a sufficient compaction of the soil in its natural condition which does not necessitate further consolidation. In the dry season, when consolidation of fills is more or less impracticable, it has been tempting to design a runway for the maximum of work in cut. All the tests made contradict this assumption and in every case, a greater density was obtainable by simple artificial consolidation, than existed in the undisturbed soil in cut. The undisturbed soil was usually of very low value indeed as compared with that needed for stability in the wet season. In almost all cases where a patch of ground was met with of a stability value rather higher than the general average, the Author was told that it was an area of fill, and extra compaction had been given."

The California Bearing Ratio of the subsoil varied from 10 to 25 (soaked) and design tables indicated that a fully compacted subgrade and base course should be obtained to a depth of at least 20 inches. Allowing for a

Figs 2

Mark	Description	Maximum dry density: lb./cubic foot	Optimum moisture content
1a	Section 6. Trial holes 16, 17, 18. Top soil	113·8	11·0%
1b	" " " " " " " Bottom layer	118·3	12·9%
2a	Section 7. Trial holes 19, 20, 21. Top soil	111·0	11·0%
2b	" " " " " " " Bottom layer	123·1	11·5%
3a	Section 8. Trial holes 22, 23, 24. Top soil	110·4	10·5%
3b	" " " " " " " Bottom layer	120·2	10·3%



Proctor Tests of Soil Samples. New Runway—Kano Airport

$7\frac{1}{2}$-inch consolidated laterite base course, it was decided that an 18-inch-thick consolidated subgrade should be provided. With the equipment available it was impossible to obtain maximum densities with soil layers more than 6 inches thick, so that if it had been arranged that finished surface levels of the new runway should approximately coincide with existing ground levels it would have been necessary to remove an additional

12 inches of soil, consolidate the exposed surface, and return the soil in 6-inch layers. To carry out this operation successfully would have required a considerable number of skilled operators and supervisors, and it was decided that the required thickness of consolidated soil could most easily be obtained by compacting the exposed surface after the removal of the topsoil and then adding two 6-inch layers of selected fill brought from the verges. This arrangement facilitated the drainage of the hardstrip, and to a large extent governed the profiles of the cross-sections adopted.

The longitudinal grade and typical cross-sections of the runway are illustrated in Figs 3, Plate 1. The runway complies in every respect with the specifications for a runway of I.C.A.O. Class 3 standard.

The climate of Nigeria, particularly that of the northern region, is markedly seasonal, and at Kano practically all the annual rainfall of about 35 inches falls within the 5 months from May to September. At the height of the rains the earth is difficult to move, but on the other hand, compaction can best be carried out when the moisture content of the soil is nearer optimum. The work was planned to make best use of the seasons, as shown on the Programme and Progress Chart, *Fig. 4*. On these charts, which are widely used in the P.W.D., the upper shading shows the dates between which it is proposed to carry out each operation, the lower shading indicates the proportion of each item completed at any particular date, and the thick horizontal line shows the dates when work actually started, and was completed or suspended. For a project of this size a large amount of mechanical earth-moving and rolling equipment is desirable. Unfortunately, Nigeria does not yet possess sufficient to cope with all such projects, and the amount necessary to mechanize the work completely could only have been concentrated at Kano at the expense of the extensive road-construction programmes in hand elsewhere. There are no well-equipped workshops near the job, skilled operators are few, and spares have been difficult to obtain. Many of the operations on which mechanical plant could have been economically employed have therefore had to be carried out manually (see *Figs 5* and *6*). Shortage and breakdowns of rolling equipment have been the most serious difficulties. In general, the earthworks were carried out by local petty contractors at a cost of less than 2 shillings per cubic yard, the labour rate in that part of the country being 1*s.* 7*d.* to 1*s.* 11*d.* per day. The total estimated cost of the project is £100,000.

The various items on the programme chart which are not self-explanatory, were carried out as follows :—

Item 6. Root holes were re-filled with selected soil in 4-inch layers, each layer being brought to optimum moisture content and consolidated by hand ramming.

Item 7. In order to provide the large amount of water necessary for consolidation it was originally planned to lay a water main with

Fig. 4

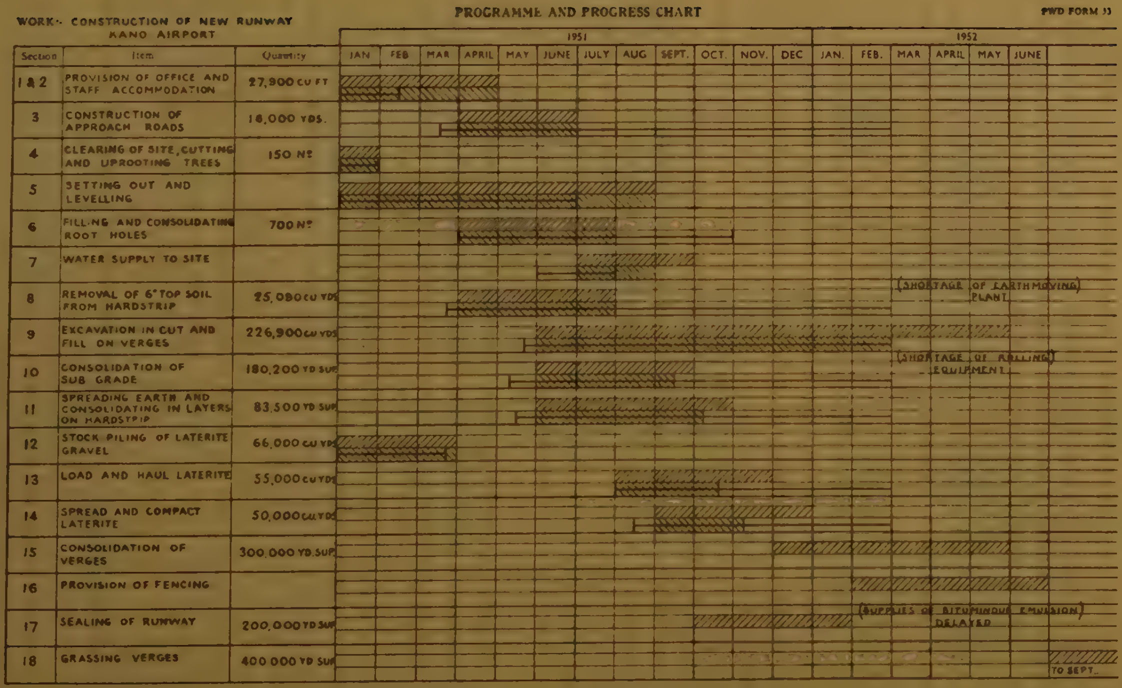

WORK:- CONSTRUCTION OF NEW RUNWAY KANO AIRPORT
PROGRAMME AND PROGRESS CHART
PWD FORM 33
Section
Item
Quantity
1951
1952
JAN
FEB
MAR
APRIL
MAY
JUNE
JULY
AUG
SEPT.
OCT.
NOV.
DEC
JAN.
FEB.
MAR
APRIL
MAY
JUNE
1 & 2 PROVISION OF OFFICE AND STAFF ACCOMMODATION 27,900 CU FT
3 CONSTRUCTION OF APPROACH ROADS 18,000 YDS.
4 CLEARING OF SITE, CUTTING AND UPROOTING TREES 150 NO
5 SETTING OUT AND LEVELLING
6 FILLING AND CONSOLIDATING ROOT HOLES 700 NO
7 WATER SUPPLY TO SITE
8 REMOVAL OF 6" TOP SOIL FROM HARDSTRIP 25,080 CU YDS
9 EXCAVATION IN CUT AND FILL ON VERGES 226,900 CU YDS
10 CONSOLIDATION OF SUB GRADE 180,200 YD SUP
11 SPREADING EARTH AND CONSOLIDATING IN LAYERS ON HARDSTRIP 83,500 YD SUP
12 STOCK PILING OF LATERITE GRAVEL 66,000 CU YDS
13 LOAD AND HAUL LATERITE 55,000 CU YDS
14 SPREAD AND COMPACT LATERITE 50,000 CU YDS
15 CONSOLIDATION OF VERGES 300,000 YD. SUP
16 PROVISION OF FENCING
17 SEALING OF RUNWAY 200,000 YD SUP
18 GRASSING VERGES 400 000 YD SUP
(SHORTAGE OF EARTHMOVING PLANT)
(SHORTAGE OF ROLLING EQUIPMENT)
(SUPPLIES OF BITUMINOUS EMULSION DELAYED)
TO SEPT.

Fig. 5

DIG AND CARRY—THE OLD WAY

Fig. 6

DIG AND CARRY—THE NEW WAY

Fig. 7

In-Situ Density Tests

Figs 9

Blinding

Above—The Old Way
Below—The New Way

hydrants at 1,000-foot intervals along the whole length of the runway. It was later found that an adequate and regular supply could not be guaranteed from the city mains and the proposal was abandoned in favour of pumping from a nearby stream into tankers.

Item 9. 50 per cent of the earthworks was carried out by local petty contractors, using either head or donkey transport. For an average carry of about 200 yards, the price was 1*s*. 9*d*. per cubic yard.

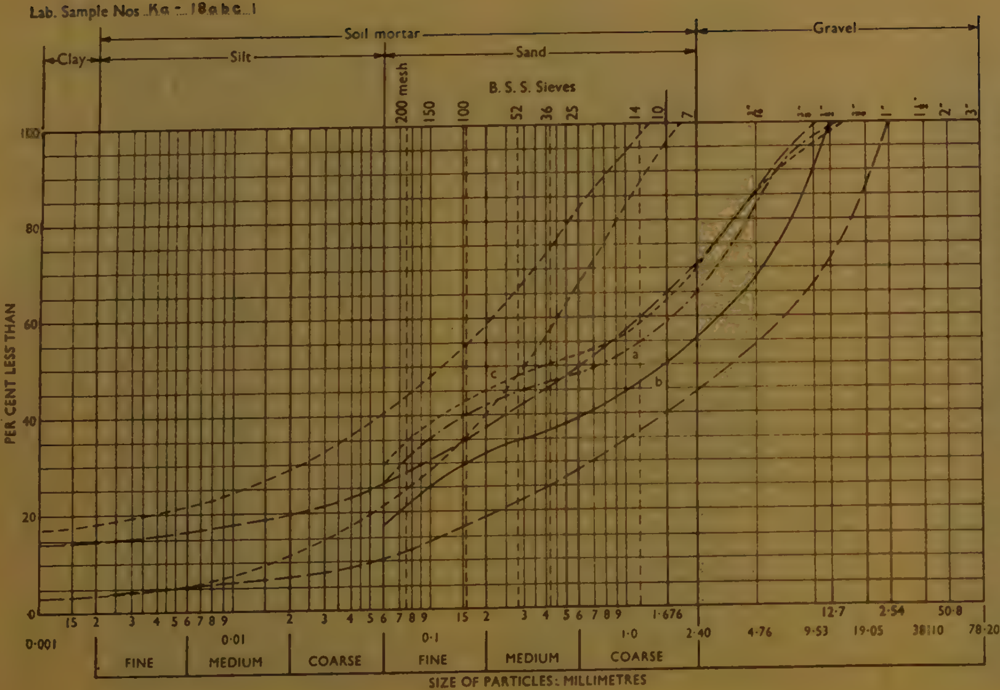
Items 10 and 11. 95 per cent relative compaction was specified for the consolidation of the subgrade, that is, a minimum dry density of 114 lb. per cubic foot at an optimum moisture content of 11 per cent. Each 6-inch layer was first watered until the moisture content, as determined by pycnometer, was about 2 per cent more than the optimum. Once the necessary specific-gravity/moisture-content curves have been obtained for a particular soil, moisture contents can be quickly determined in the field by pycnometer. The method is susceptible to small errors resulting from changes of temperature in the water used, but these can be kept within reasonable limits if the calibration is periodically checked by comparison with results obtained from oven-dried samples. When the soil was sufficiently wetted, rolling was commenced and continued until densities (determined by the core-cutter method (see *Fig. 7*)) were obtained higher than the specified minimum. It was generally found that adequate compaction could be obtained with fifteen passes of a 10-cwt vibratory roller and ten passes of a 6-ton roller.

The relative compactions obtained varied from 95 to 103 per cent, the average being about 97·5 per cent. In a very few isolated spots the figure was 94 per cent, but it was considered that the factor of safety and the 7½-inch laterite base course would give sufficient protection.

Because of the shortage and breakdown of rolling equipment, hand ramming had to be employed at times, but the thickness of the soil layers had to be reduced to about 4 inches. This was naturally more expensive than power rolling, but there was no alternative and the results were just as good. On one occasion, hand ramming had to be adopted for about 3 months, after which fully-laden 5-ton Albions were used.

Items 12, 13, and 14. Adequate quantities of laterite are available from three deposits situated near the runway, as shown in Fig. 10, Plate 2. Results of laboratory tests on three typical samples of these are shown in *Fig. 8*, and a minimum dry density of 120 lb. per cubic foot was specified, that is, about 100 per cent relative compaction. Consolidation was carried out in a manner similar to that employed on the subgrade, except that the thickness of the layers was reduced to 4½ inches and densities were determined by the sand-bottle method. Relative compactions of about 110 per cent were obtained.

Fig. 8



SOIL ANALYSIS CHART
P.W.D. No. 24401

MATERIAL PARTICULARS

Origin: KANO AIRPORT
Purpose: CLASSIFICATION (NEW RUNWAY)
Reference

Lab. sample No. KA18	a	b	c
Appearance	Brown	Dark Brown	Light Brown
Exact location	1	2	3
Gravel % of total	33	44	30
Sand " " "	41	37	40
Silt " " " } Clay " " "	26	19	30
Liquid limit %	27	25	28
Plastic limit %	21	19	24
Plasticity index	6	6	4
Opt. moist. content %	13·0	12·0	13·5
Max. dry density lb./ft³	122·0	125·0	118·0
Density in situ lb./ft³			
S. G. of particles			
S. G. of silt and clay			
Classification	A·2·4	A·1·b	A·2·4
Group index	0	0	0

Analysis by Lab. Date 18·11·51

Remarks

Signed

LATERITE SAMPLES. KANO AIRPORT

Item 17. The sealing specification is M.C.I., cutback for the first coat and bitumen emulsion for the second coat. The work is in hand and good penetration on the tightly knit laterite surface has been obtained with the M.C.I., blinded with gravel at about 80 square yards per cubic yard (*Figs 9*). A gang of 48 men can complete 4,200 square yards of single-coat work in a day. The work is more fully described under Ikeja. New taxistrips are to be constructed as shown in Fig. 10, Plate 2, to a similar specification.

The following items of plant were used for the various operations, when contractors' labour was not employed :—

One M.12 grader and one D.12 grader for initial levelling of the natural ground and removal of initial layer of 6 inches of top soil.

Four D.6 caterpillar scrapers for excavation on one verge and carrying to fill on opposite verge, and laying of base courses on the hard strip.

Two graders for levelling each base course after the material had been deposited by scrapers.

Two D.6 bulldozers for working with the scrapers on verges. These machines broke up the soil, which was extremely hard, especially where a mixture of laterite gravel was encountered, for subsequent picking up by scrapers.

Two vibratory rollers and two 6-ton rollers (when available) for compaction of subgrade and base courses. In the absence of the heavy rollers, two 2½-ton rollers were used after initial compaction by 5-ton tipping lorries.

Three 6-ton rollers for compaction of laterite gravel carpet and surfacing.

One grader for blading top of base course before depositing laterite gravel and for blading top of laterite carpet preparatory to laying first coat of tar.

One 2½-ton roller for rolling gravel and sand blinding.

Two Muirhill loaders, one ¼-cubic-yard Priestman shovel, one D.6 bulldozer, and six 5-ton Albion tipping lorries for loading and hauling laterite gravel.

Construction dates were as follows :—

Collection of laterite, January to March 1951 ; 66,000 cubic yards.

Actual construction commenced in May 1951.

Consolidation of subgrade and base courses completed in April 1952.

Consolidation of laterite carpet in two layers expected to be completed by May 1952.

Sealing commenced in March 1952, and expected to be completed by June 1952.

IKEJA AIRPORT.

Ikeja, the airport of Lagos, is situated in flat country 130 feet above sea-level and 11 miles north of the city. The airfield was constructed in 1941 with two runways, 1,800 and 2,000 yards long. No. 1 runway has a $1\frac{1}{2}$- to 2-inch bituminous premix carpet laid on a laterite base course varying in thickness from $1\frac{1}{2}$ to 2 inches. No. 2 runway was the natural soil, graded, well-consolidated, and grassed; at certain periods during the wet season, it had to be closed. Generally speaking, the soil is a reasonably well graded sandy-clay classification A-4. (See *Fig. 11*, Sample C.)

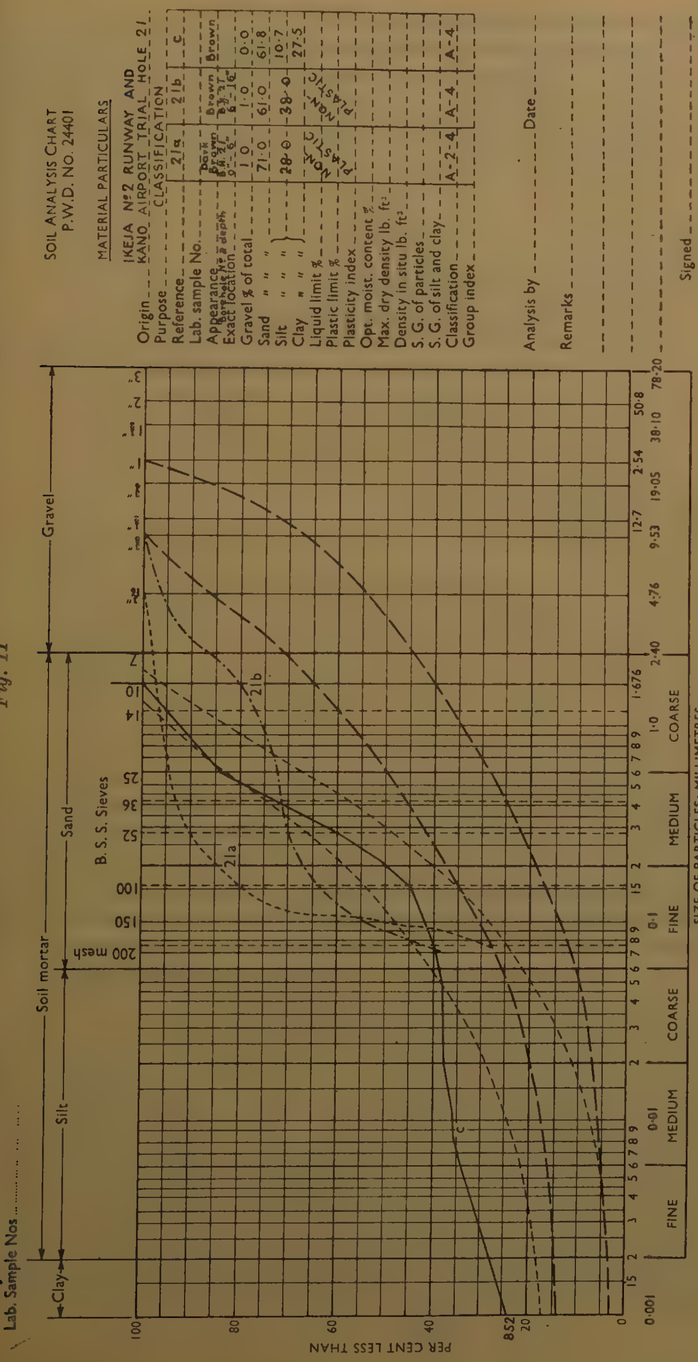
In 1949 the Air Ministry carried out load-bearing tests, in which uniform incremental loads were applied to circular steel plates. At each test point, cycles of loading were applied four times in equal stages up to contact pressures of 45 lb. per square inch. This was then repeated for higher contact pressures and finally the load was increased until failure occurred or until the limit of the testing apparatus was reached.

The deflexions at each increment of loading were recorded.

Tests were carried out using plates of 30, 26, and 15 inches diameter on the surface and subgrade of No. 1 runway and at five points along the centre-line on the surface of No. 2 runway. The tests on No. 2 runway gave uniformly good results. When using a 26-inch diameter plate, failure did not occur within the limits of the testing machine (70,000 lb. or 132 lb. per square inch). The average load causing a deflexion of 0·10-inch for the final load cycle after one repetition of load at a contact pressure of 60 lb. per square inch, was 54,000 lb. (102 lb. per square inch). These tests were carried out in the dry season and it was found that under these conditions the bearing capacity of No. 2 runway was in fact superior to that of No. 1; it was estimated that with adequate compaction at suitable moisture content, safe bearing capacities of 90 lb. per square inch could be achieved. These tests confirmed the original proposal that Ikeja could be best and most economically provided with a new runway of suitable dimensions and high bearing capacity by the extension and reconstruction of the existing No. 2 runway.

It was decided to extend the runway 500 feet northwards and 100 feet southwards and provide a hardstrip 2,200 yards long and 150 feet wide, comprising a laterite base course with a minimum thickness of 4 inches, and a bituminous surface dressing. Before construction started, in-situ density tests were carried out at surface level and 10 inches depth over the whole of the existing runway area, and it was found that in two areas, totalling 1,100 feet in length, existing densities were comparatively low. The runway surface was first scarified and the top soil removed to a depth of $2\frac{3}{4}$ inches. Where the soil was of low density a further 8 inches of soil was removed and the exposed surface compacted; the upper layer was

Fig. 11



MECHANICAL ANALYSIS OF SOIL SAMPLES

No. 2 RUNWAY—IKEJA AND NEW RUNWAY—KANO

then replaced, additional fills being brought from the verges where necessary. The whole area was then compacted at surface level. The rolling procedure used for compaction in each case was twenty-one passes of a sheepsfoot roller followed by two passes of a vibratory roller and four passes of a 6-ton roller. Average results of density tests carried out before and after compaction were as shown in Table 1.

TABLE 1

(1). **Areas compacted at surface level only.**

Depth	Dry densities : lb. per square foot	
	Before compaction	After compaction
0″–5″	112·5	117·3
10″–15″	100·0	102·8

(2). **Areas compacted in two layers.**

Depth	Dry densities : lb. per square foot		
	Before compaction	After compaction of bottom layer	After final compaction
0″–15″	110·3	—	109·6
10″–15″	96·6	107·7	110·9

In its existing state the ground had been well compacted by the action of traffic during the previous 9 years, and where densities were high initially, no great increase could be expected. Rolling at surface level had little effect on the soil at depths greater than 10 inches. Suitable laterite was not available in the immediate vicinity of the airport, and all laterite necessary for the work was supplied by contractors from deposits 5 miles away. At least two passes of a vibratory roller followed by two passes of a 6-ton roller were used for consolidation.

The hardstrip was sealed with two coats of bituminous emulsion (0·35 and 0·22 gallons per square yard) and blinded with sand (122 square yards per cubic yard), after the laterite had been brushed, wetted, and finally rolled with one pass of a 6-ton roller. The emulsion was spread by hand, about 50 per cent water being added to the first coat to enable a uniform covering to be more readily obtained, and each coat was rolled with four passes of a 6-ton roller a few hours after blinding. The hardstrip was divided into strips about 21 feet wide and work proceeded from one strip to the next but one; a gang of 70 men were able to complete 5,400 square yards of single-coat work per day. The runway had to be

put into operation soon after sealing was completed, and to prevent the runway markers being obliterated, all loose sand had to be brushed off the runway somewhat sooner than was desirable.

Thirty acres of bush had to be cleared for the runway extensions and necessary overruns, and considerable earthworks were necessary to bring the verges to required formation levels consistent with the design of the existing runway. Compaction of the subsoil on the hardstrip area of the extension was carried out at 18 and 9 inches depth and at surface level by a similar rolling procedure as that used for the remainder of the runway.

The P.W.D. central workshops are at Lagos and mechanical plant for the work was more readily available and more easily maintained than was the case for the Kano construction. The following items of plant were used for the various operations, though they were not always available, owing to breakdowns and other high-priority works :—

Two M.12 graders for initial scarifying and grading of laterite base course and verges.

Three D.6 bulldozers for clearing bush and earthworks on extensions and handling of soil layers in areas where additional compaction was applied.

One sheepsfoot roller and one vibratory roller with tractor.

Two 6-ton rollers for compaction of sub-soil, base course, and surfacing.

Six 2-cubic-yard dumpers with three Muirhill loaders for earthworks, transport of sand, and laterite.

The relative costs of the various operations are given in Table 2. These figures do not take into account any proportion of the original cost or hire charges for the plant used. Supervision charges from divisional and central headquarters and the salaries of the engineer in charge of the work and the African technical staff engaged on surveying and soil testing are also not included. Wages at Ikeja are 4*s*. 6*d*. to 9*s*. 0*d*. per day for plant operators, etc., and 2*s*. 6*d*. for unskilled labour. Diesel oil costs 2*s*. 0*d*. per gallon and petrol 2*s*. 8*d*. per gallon.

Construction dates were as follows :—

Laterite was collected during 1949. Actual construction started in April 1950, and consolidation of the subsoil and laterite base course was completed by the end of the wet season in October. 17 inches of rain fell at Lagos in June 1950, when excavated areas were flooded and progress on the work was delayed for about 3 weeks. Sealing was started when the runway had dried out in December, and was completed in 12 weeks. The runway was opened to traffic in April 1951, and has been in continuous use since that date. No serious defects have been apparent, but slight wheel-tracking has been caused by the quick turning of aircraft when ground temperatures have been sufficiently high to soften the bituminous surface.

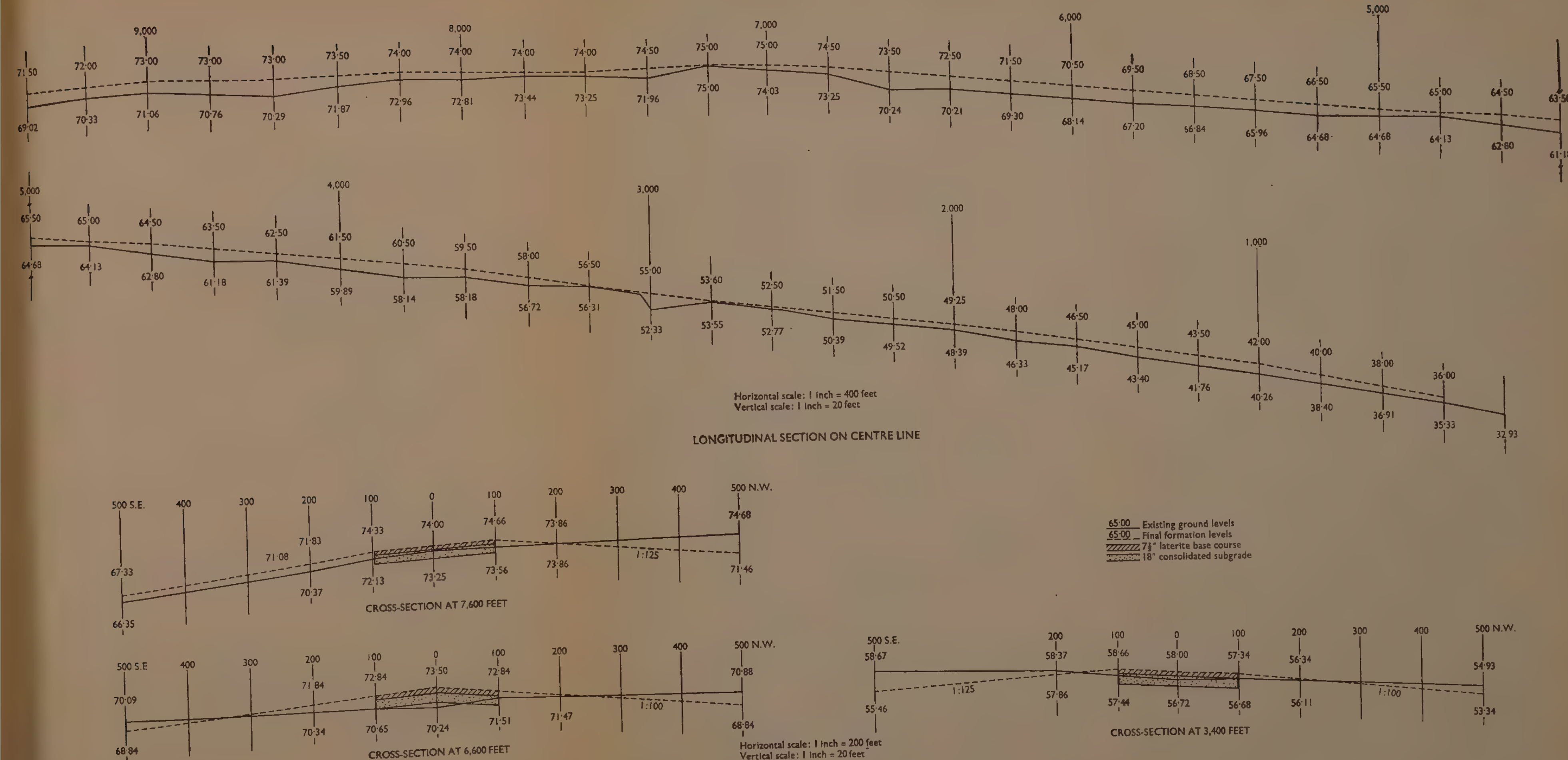
TABLE 2.—SUMMARY OF COSTS FOR RECONSTRUCTION OF 1,900 YARDS OF RUNWAY (95,000 SQUARE YARDS) WHERE FINAL LEVELS WERE APPROXIMATELY EQUAL TO ORIGINAL FORMATION LEVELS.

No.	Item	Cost per sq. yd: d.	Total cost: £	Cost per sq. yd: d.	Total cost: £
1	Surveying and preliminaries . .			0·4	158
2	Scarifying			0·07	28
3	Removing 2¾″ top soil and levelling			0·478	189
4	Additional compaction on areas of low initial density at 8″ below surface level. 18,333 square yards, including excavation, all rolling and return of soil			0·5812	444
5	Grading part area to formation levels after sheepsfoot rolling .			0·055	21
6	Compaction of subgrade. Average of :—				
	21 passes with vibrating rollers .	0·293			
	3 ,, ,, vibrating rollers .	0·1776			
	5 ,, ,, 6-ton roller . .	0·309			
				0·7796	319
7	Provision and spreading of laterite, 13,300 cubic yards at 10·3*d.* per yard (average 5″ thick) . . .	17·220	6,816		
	Labour	1·173	464		
				18·393	7,280
8	Grading of laterite			0·55	213
9	Compaction of laterite : average of :				
	5 passes with vibrating rollers .	0·2906			
	5 ,, ,, 6-ton roller . .	0·309			
				0·600	232
10	First coat sealing :				
	33,200 gallons of bitumen at 24·534*d.* per gallon . .	8·585	3,398		
	776 yards of sand at 8*s.* per cubic yard	0·786	311		
	Labour and tools	0·9174	363		
	4 passes of 6-ton roller . . .	0·247	98		
				10·5354	4,170
11	Second coat sealing :				
	23,900 gallons of bitumen at 24·534*d.* per gallon . . .	5·40	2,138		
	776 yards of sand at 8*s.* per cubic yard	0·786	311		
	Labour and tools	0·9418	372		
	4 passes of 6-ton roller . . .	0·247	98		
				7·3748	2,919
12	Removal of surplus sand from surface			0·1155	46
13	Patching and correcting of surface			0·3870	153
14	Sundry works, daily-paid office staff, etc.			0·4536	180
15	Final works, provision of markers, etc.			0·568	225
	Total for 95,000 square yards .			41·44	16,419

Notes to the above Table are given on top of page 187.

AERODROME RUNWAY CONSTRUCTION IN NIGERIA

FIGS 3

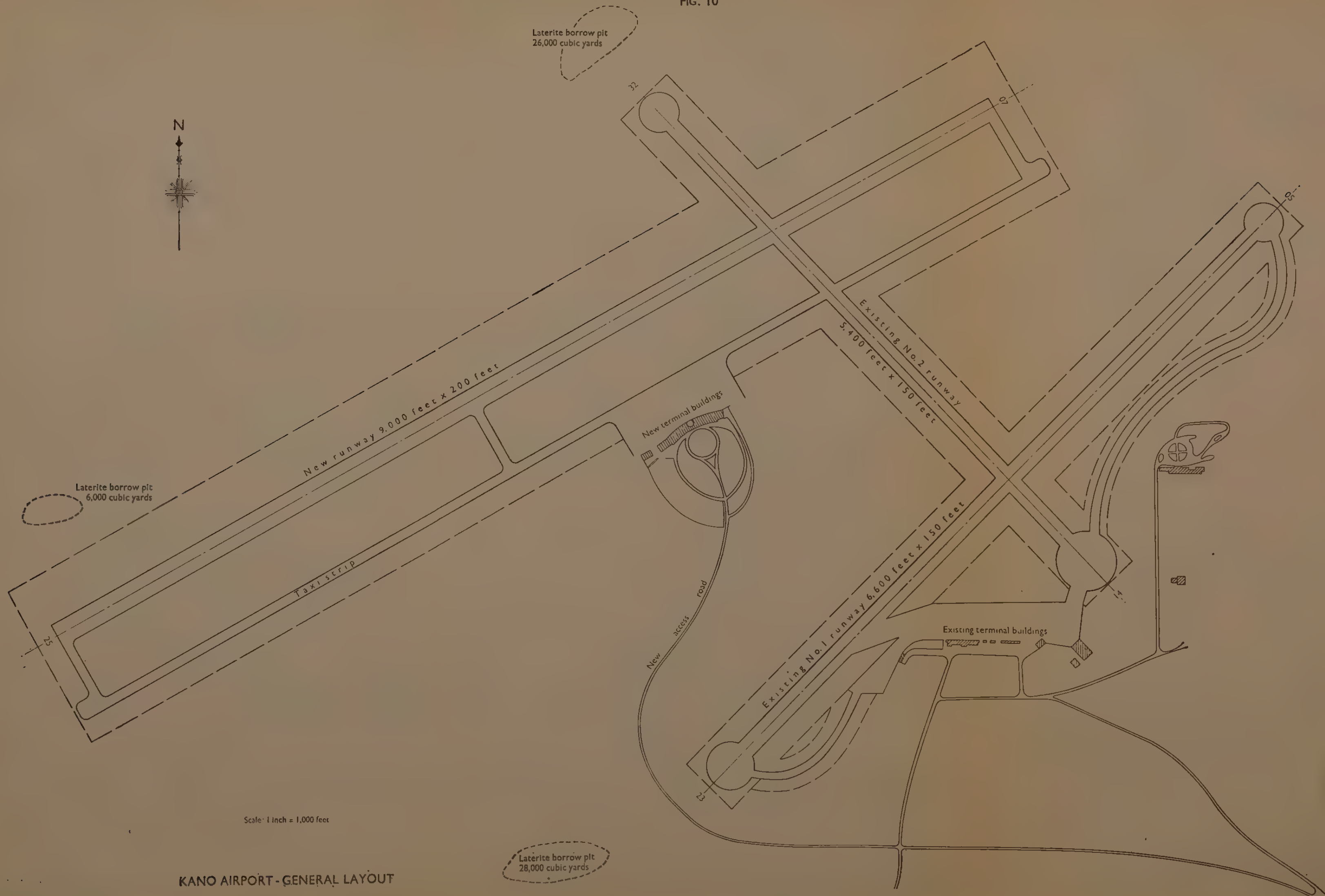


KANO AIRPORT - NEW RUNWAY

ALEXANDER McDONALD

AERODROME RUNWAY CONSTRUCTION IN NIGERIA

FIG. 10



KANO AIRPORT - GENERAL LAYOUT

The reconstruction of the end 100 yards of runway to increased formation levels to conform with the designs adopted for the extensions and the construction of 200 yards of new runway.

Cost = £3,635 ; at 52·333*d*. per square yard.

The laying of 1,667 yards of premixed bituminous carpet at the intersection with the existing No. 1 runway cost £295.

Total cost for whole work = £20,349.

ACKNOWLEDGEMENTS

The material for the Paper was collected by Mr G. A. Bell, B.Sc.(Eng.), Public Works Department, Nigeria, to whom grateful acknowledgement is made. Thanks are also due to Mr M. R. Carr-Hall, B.Sc.(Eng.), who was in charge of Kano construction and in-situ tests, and who has checked the data for the Kano runway.

The Paper is accompanied by fourteen photographs and eight sheets of diagrams, from some of which the half-tone page plate, Folding Plates 1 and 2, and the Figures in the text have been prepared.

Paper No. 5891

"Sea Defence Work at Silloth, Cumberland"

by

Geoffrey Owen Lockwood, M.C., M.I.C.E.

(*Ordered by the Council to be published with written discussion.*)†

SYNOPSIS

The Paper describes the construction of sea defence work at Silloth, about ½ mile in length, and consisting of a framework of steel sheet-piling covered by an in-situ-concrete stepped apron supported on imported filling, together with the driving of eight home-grown elm timber groynes, each 200 feet long, all work being carried out in tidal waters.

The concrete stepped apron, which has fifteen 3-foot-wide and two 6-foot-wide treads, with slopes ½ inch in 1 foot and 10½-inch risers, is reinforced top and bottom with square-mesh reinforcement; all concrete was vibrated.

A 15-foot-wide concrete flagged footpath is laid at the top of the apron as a further protection from wave splash, and the graded ground back to the existing green is turfed or sown with grass.

The completion of the scheme required 1,050 tons of steel sheet-piling (of which 450 tons in the toe of the apron consisted of copper-bearing steel) and 9,000 cubic yards of vibrated reinforced concrete in the stepped apron, and 60,000 cubic yards of imported filling. The estimated cost of the complete work was £154,000.

HISTORY

IN presenting this Paper on the design and construction of the sea defence works recently completed at Silloth, it has been thought that it might be helpful if a short account were given of the history of the conditions which led to the combined Local Authorities undertaking these sea defence works.

In 1847, Silloth consisted of a small hamlet made up of farms all in one ownership. A company called the Carlisle and Silloth Bay Railway and Dock Company was formed in 1854, and after obtaining Parliamentary powers in 1855, the railway connecting Silloth with Port Carlisle (which already had a railway connecting it to the City of Carlisle) was completed and opened in 1856.

The construction of the first dock was commenced in 1857 and completed in 1859. Much of the excavation for this dock and a second dock, completed in 1885, was spread on the adjoining low-lying sandhills, and on this levelled area about 36 acres of open green were laid out to form the sea front to the town.

† Correspondence on this Paper should be received at the Institution by the 1st June, 1953, and will be published in Part II of the Proceedings. Contributions should be limited to about 1,200 words.—SEC. I.C.E.

Adjoining the docks, the Company also constructed a pier consisting of a timber piled and braced structure extending for 1,000 feet seawards. This pier (which has since been shortened by about 200 feet) interferes with the free flow of waterborne sand and gravel in a north-easterly direction, thereby tending to starve the beach fronting the green. This tendency, together with the damage caused by the combination of high tides and north-westerly gales, soon rendered coast protection measures necessary for about $1\frac{1}{2}$ mile on the north side of the timber pier.

The revetment in early days took the form of timber sheeting backed with boulders, and timber groynes, but the sea continued to encroach, although repairs were always effected after storm damage.

In due course the North British Railway Company took over the docks and sea front from the Carlisle and Silloth Bay Railway and Dock Company, and over the years these sea defences immediately north of the dock, for a length of about $\frac{1}{2}$ mile, were strengthened in various forms, until finally they were substantially strengthened by the London and North Eastern Railway Company with railway metals and sleepers built in the form of a stockade, which was filled with sand and gravel and sealed off with a concrete top, together with freestone pitching at the toe.

These defences, however, suffered considerable damage through storms in the early part of the 1939–1945 war, and in 1944 the Railway Company informed the Wigton Rural District Council that, since the cost of the repair work had been considerable, with no resultant benefit to the Company, they had decided that they could no longer continue to maintain the sea defence works in front of the green except at points where such defences were necessary to protect their dock property. In consequence, the Rural District Council consulted the County Council as to the best course of action to take. The Rural District Council and the County Council were both interested from a planning standpoint in preserving the amenities of the green as being vital to Silloth as a seaside holiday resort for Cumberland.

The Ministry of War Transport were first approached, since the sea defence works immediately north of the green are in close proximity to a classified road and the Ministry accept the cost of repair for classification grant purposes owing to their being necessary for the maintenance of the road. They could not agree, however, that the section fronting the green, where the beach is further from the road, could be similarly accepted.

Negotiations with the Ministry of Health were then entered into and a detailed report and estimate of the works required were submitted in March 1947. This report made reference to the Prime Minister's statement in the House of Commons on the 27th January, 1947, to the effect that the Government had decided that the general responsibility for dealing with problems of coast protection should, in England, rest with the Ministry of Health; the powers and duties of county district councils

were to be increased, and county councils were required to make suitable contributions towards the costs of approved schemes.

The Ministry of Health indicated that, while the land to be protected remained in private ownership, no grant was possible. A deputation from the two Councils waited on the Ministry of Health in May 1947, and following this meeting, the County Council decided to undertake the scheme of sea defence works described in this Paper in conjunction with the Wigton Rural District Council, with the County Engineer designing and supervising the works. Negotiations took place with the Railway Company whereby the Green was transferred to the ownership of the Rural District Council for a nominal consideration.

Further extensive storm damage had now made the provision of new defence works a matter of urgency (see *Fig. 3*), and the submitted scheme, following an informal Inquiry in January 1948, was eventually approved by the Ministry of Health after consultation with the Ministry of Transport regarding works below high-water mark.

Tenders were invited in September 1948 and the accepted contractors, Richard Costain Ltd, commenced work on the site on the 1st March, 1949, and completed the scheme in 2 years.

The Ministry of Health made a 60-per-cent grant towards the cost of the works and the 40-per-cent balance of expenditure was shared, two-thirds by the County Council and one-third by the Wigton Rural District Council, the estimated cost of the works, including administrative charges, being £154,000.

General Description of Works

The site of the works is on the foreshore of the Green and extends 800 linear yards from the dock property at the old lifeboat house, to join up with the existing concrete defence works farther north (see Fig. 1, Plate 1).

The works involved the driving of a front row of steel sheet-piles, having a minimum penetration into the beach of 15 feet, a back row of steel sheet-piles 57 feet behind the front row at a higher level, and cross-rows of rising piles at 60-foot centres, locked into the front and back rows, the whole structure forming a series of boxes to localize any future storm damage, as shown in *Figs 4* and *5*. A bold line was chosen for the defences, thus restoring extensive areas where erosion had taken place, and this necessitated considerable imported filling in the boxes over the greater part of the length of the works. Concrete pads were formed on the heads of the cross-ways of piling and a 1-foot-6-inch-thick reinforced vibrated-concrete stepped apron was constructed to protect the filling. This stepped apron has fifteen 3-foot-wide and two 6-foot-wide treads and $10\frac{1}{2}$-inch risers, and is shown in Figs 2, Plate 2, and *Figs 5* and *6*. The treads have a slope seawards of $\frac{1}{2}$ inch in 1 foot and the riser faces are at

Fig. 3

CENTRE VIEW BEFORE COMMENCEMENT OF WORK, SHOWING EROSION AND STORM DAMAGE

Fig. 4

CENTRE VIEW, SHOWING STEEL SHEET-PILING IN FRONT WITH STEPPED CROSS-PILING AND COMMENCEMENT OF FILLING

Fig. 5

View from South End, showing Concrete Stepped Apron under Construction

Fig. 6

View from South End, showing Completed Work

right angles to the treads. This small seaward inclination of the risers has been very successful in controlling the effects of wave splash. The level of the bottom step is 10·00 A.O.D. and the top step 26·50 A.O.D.; contraction joints were formed at 60-foot centres and expansion joints at 300-foot centres; two ramps to permit access to the beach were also formed.

Equinoctial spring tides reach 17·50 A.O.D.; the highest recorded tide (in 1902) was 20·75 A.O.D., and during construction the highest tide experienced reached 19·00 A.O.D.

At the top of this concrete stepped apron a 15-foot-wide concrete flagged footpath was laid to a slope of 1 in 48 as a further protection from wave splash. The ground was then graded-back to the existing green, the area being sown-down, with the exception of a steep bank which was retained as a feature and laid with Silloth turf. Eight timber groynes, 200 feet long, were driven at 300-foot centres and at right angles (considered the best angle for this beach) to the general line of the concrete stepped apron to maintain and build-up the level of the beach. These groynes consisted of home-grown elm piles 9 inches square at 6-foot centres, driven to a minimum penetration into the beach of 9 feet, and were planked progressively with 9-inch-by-3-inch elm planks in 12-foot lengths as the beach built up (see Figs 2, Plate 2). All timber work was impregnated under vacuum and pressure with "Celcure" wood preservative with an average net absorption of ½ lb. of dried Celcure salt per cubic foot of timber.

The works also involved the construction of two deep manholes and the laying through the defence works of a 21-inch-diameter cast-iron sewer at a lower level on the line of the removed 18-inch-diameter main outfall from the town, as well as a 15-inch-diameter cast-iron stormwater overflow. This main sewer reduces from a 21-inch-diameter to an existing 9-inch-diameter sewer on the seaward side of the front row of piles and extends to below low-water mark, the pipes being relaid and supported on cradles fabricated from 75-lb. steel rails recovered from the previous defence works. The 21-inch-diameter pipe was laid to allow for any future development of the town to take place without disturbing the defence works. Further pipe lines laid through the works include a 6-inch-diameter cast-iron rising main to a proposed swimming pool, and a 6-inch-diameter cast-iron outlet from a proposed paddling pool.

The completion of the scheme necessitated some 450 tons of copper-bearing-steel sheet-piling in the toe wall, 600 tons of mild-steel sheet-piling in the back and cross-walls, 9,000 cubic yards of vibrated reinforced concrete in the stepped apron, and 60,000 cubic yards of imported filling.

The docks end of the works is protected by a mass concrete retaining wall 17 feet high which returns into the line of the existing timber defence wall protecting the docks property. At the time, this timber wall was considered to be in a dangerous state of disrepair and provision was made for extending the concrete wall, and these works are now in hand following

further deterioration of the timber structure. The basis of apportionment of cost is similar to the previous works, the estimated cost being £83,000.

Construction

General

The site offices were established in a disused building immediately behind the mid-point of the works, having easy access to the highway. An area of shrubs was cleared and hardstanding laid down to provide for a plant yard, steel-benders' benches, and slab-casting yard, the central concrete batching and mixing plant, and three aggregate storage bins.

A derrick track was laid the full length of the works at top level and a Jubilee track from the concreting plant to the extreme limits of the works.

Parts of the beach where the existing "sleeper and rail" defences had been breached were covered with twisted railway metals and slabs of concrete. It was necessary to clear this old work so that a start could be made driving steel sheet-piles. Approximately 2,500 linear yards of rails were recovered, 500 cubic yards of concrete and boulders broken-up and moved out of the line of the works (to be incorporated as filling at a later stage), and 4,800 cubic feet of old timber piles from previous defence systems withdrawn.

In order to prevent further encroachment of the sea, it was necessary that the works be carried out in a specific order, the first stage consisting of the driving of the steel sheet-piling in the toe wall, the second the construction of the groynes, and finally filling behind the toe wall to pile head level.

Piling

Three boreholes sunk in the line of the toe wall indicated that sand and gravel would be met with in driving the piles to a depth of up to 25 feet below existing beach level, and test piles driven in the line to a depth of 15 feet proved this to be so.

The type of piling chosen was Appleby Frodingham No. II section, weighing 24·17 lb. per square foot of wall, all piles in the front wall (and wherever subject to tidal action) having a 0·35-per-cent copper content, and varying in length from 15 to 22 feet. Driving was carried out using two sets of equipment; from approximately the halfway-stage north, using a 19 RB on the beach, and from the south end towards the north using a 3-ton electric-motored derrick with a 90-foot jib, at top level. Two No. 7 McKiernan-Terry hammers were used, one driven by steam from a No. 12 Spencer-Hopwood boiler, the other operated by compressed air from two compressors coupled to a reservoir. The top guide rails for the piles were constructed from 12-inch-by-12-inch timbers sufficiently long to seat at one end on two double piles, and allow ten pairs of piles to be pitched, with sufficient tolerance at the other end to rest on a piling frame consisting of two 12-inch-by-12-inch vertical timbers cast into a concrete base with a

SEA DEFENCE WORK AT SILLOTH, CUMBERLAND

FIG. 1

PLAN OF SILLOTH SHOWING SITE OF SEA DEFENCE WORKS

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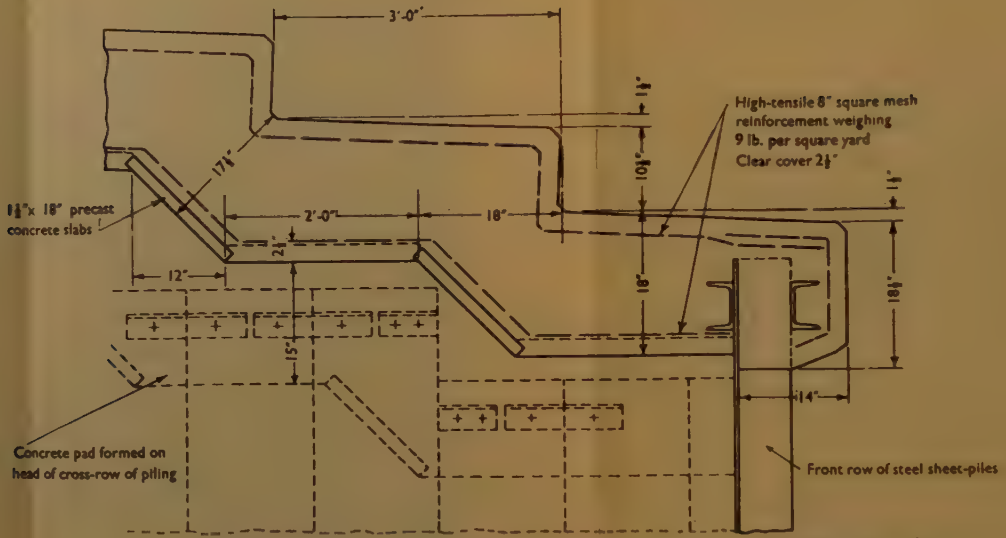
G. O. LOCKWOOD

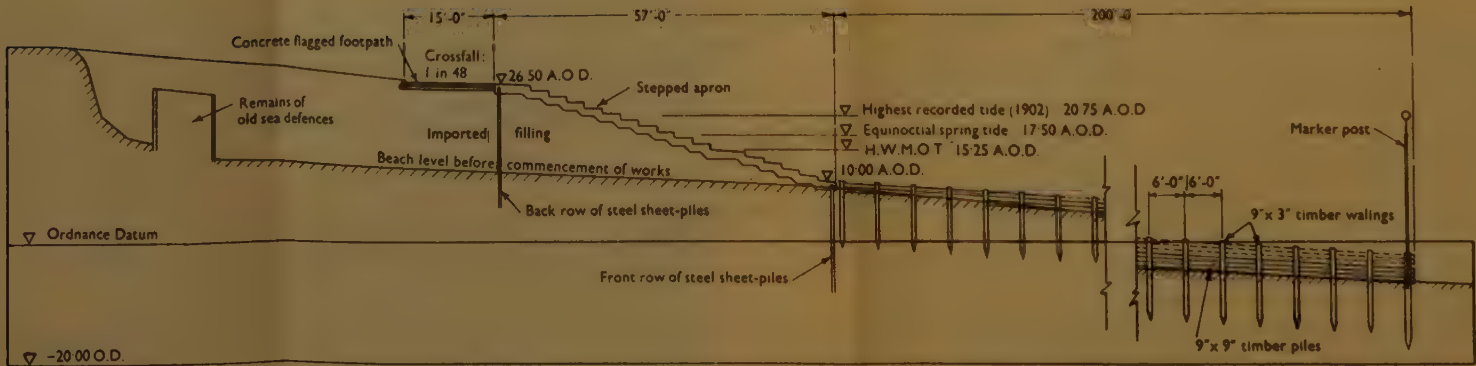
PLATE 2
SEA DEFENCE WORK
AT SILLOTH

SEA DEFENCE WORK AT SILLOTH, CUMBERLAND

FIGS 2



STEPPED APRON AT CROSS-PILING WALL



TYPICAL CROSS-SECTION

G. O. LOCKWOOD

The Institution of Civil Engineers. Proceedings, Part II, February 1953

William Clowes & Sons, Limited: London

12-inch-by-12-inch horizontal timber lance-piece 3 feet long, approximately 8 feet above the base level. A bottom guide rail, consisting of a 12-inch-by-12-inch timber strengthened with two lengths of 75-lb. railway metal fixed at beach level, ensured that the piles having been pitched to line were driven plumb throughout their length. In view of the box-shaped construction it was important that the piles did not creep, and a constant check against this was kept while driving was in progress.

Similar procedure was adopted for the back row and cross-rows of piling which varied in length between 6 feet and 23 feet, and no real difficulty was experienced in driving the cross-walls between the junction piles in the front and back rows although there was a tendency for the pile clutches to " spring " when wedging against creep.

All pile-driving was carried out between tides and it was necessary, except at neap tides, to remove all equipment from the beach.

The toe wall was completed in 5 months, and in time to check further erosion by the autumn tides. The heads of the piles were fitted with a continuous channel waling of 7 inches by 3½ inches by 18·28 lb. per foot section fixed to both faces with ⅝-inch-diameter bolts ; this waling also forms a key for the concrete apron which extends 7 inches in front of the face of the toe piles (see Figs 2, Plate 2).

Filling

As the toe piling proceeded and cross-walls were driven, a start was made importing the sand filling from the docks excavation spoil heap about 1½ mile distant. Haulage was by 5-cubic-yard tipping lorries, the majority of them being four-wheel-drive vehicles. These proved to be ideal over the soft ground although at times they had to be assisted by a bulldozer. All filling was "end " tipped and dozed into position in approximately 12-inch layers, the dozer making several passes to effect compaction. This method, whilst practical behind the defence wall, could not be used within the boxes and here the following methods were adopted.

Filling was placed proud of finished level at the respective steps in 12-inch layers using an excavator fitted with a grab. Up to the 15·00 A.O.D. level all filling was covered by the sea at normal high tide. When the tide had receded to below pile head level at 9·50 A.O.D., steps having a 2-foot-wide tread and a sloping riser face of 1 to 1 were formed by hand and rammed with hand punners to finished level. Precast concrete slabs 2 feet long and 1½ inch thick were then laid and jointed on the sloping faces and the treads formed with in-situ concrete 2½ inches thick between adjacent cross-walls, " 417 " cement being used in the lower steps. During a normal tide it was possible to construct three sets of steps over a length of 60 feet. Immediately following the next tide, the reinforced concrete apron was constructed on this base course.

Above the 15·00 A.O.D. level, alternative means of compaction of the filling were necessary. Filling was placed as before but was thoroughly

watered using 2-inch and 3-inch centrifugal pumps and seawater, and consolidation effected by the ponding method.

A heavy timber base was fitted to the No. 7 McKeirnan-Terry hammer as an alternative method to compact the filling, but the rate of vibration, approximately 180 blows per minute, was inadequate and the method was abandoned.

Sixty thousand cubic yards of selected filling were imported and compacted to complete the works. Large boulders and broken concrete from the beach were incorporated but at no time were the contractors permitted to remove material from the beach, this practice being strictly forbidden. Inspection chambers fitted with water-tight covers have been provided at intervals through the finished concrete apron so that a check can be kept on the filled area, and no movement of the filling has taken place to date.

In one section, the sea had encroached into a bank on the foreshore, about 50 feet high (see *Fig. 3*), and to stabilize this it was necessary to reduce the existing slope to 1½ to 1. This was done by dumping hardcore for a depth of about 10 feet and a width of 15 feet at the toe, and spreading and consolidating sand filling with a bulldozer operating along the line of the bank. This area was later laid with Silloth turf, about 1,200 square yards being used.

Concrete Stepped Apron

The 18-inch-thick vibrated-concrete stepped apron was designed as a protection only, and depends for its stability on the retention of the compacted filling underneath. It is reinforced top and bottom between cross-piling walls to prevent cracking, with a high-tensile square-mesh reinforcement weighing 9 lb. per square yard, the reinforcement having a clear cover of 2½ inches on exposed faces. A gravel aggregate was used, partly rounded and partly angular, of a maximum size of 1½ inch, carefully graded to requirements, the concrete consisting of a 5·75 to 1 mix by weight. Considerable care was taken in the grading and batching operations and the water/cement ratio was carefully maintained at 0·43, which gave a workable mix, with a slump of 1½ inch.

The central concrete plant consisted of a 42/28 non-tilt mixer with a potential output of 34 cubic yards of mixed material per hour, adequately equipped with storage hoppers for aggregate and sand.

The concrete placed in the bottom six steps, namely up to 15·00 A.O.D., was mixed with "417" cement; between the sixth and eleventh step up to 20·00 A.O.D. it was mixed with rapid-hardening cement, and above this level with normal Portland cement.

The shuttering was specially manufactured for the work from $\frac{3}{16}$-inch-thick mild-steel plates in 8-foot sections, covering three steps at a time for a length of 60 feet between cross-piling walls. When assembled, these

sections allowed sufficient latitude to maintain smooth curves on the curved portions.

All concrete was vibrated in place by immersion-type vibrators capable of a rate of not less than 5,000 cycles per minute, and having a $2\frac{1}{2}$-inch-diameter "poker."

The concrete was placed in the bays between cross-piling walls on the "hit-and-miss" principle, it being found possible to concrete a maximum of three steps in the lower levels between tides, and allow a sufficient time for hardening to prevent scrubbing action by the flowing tide.

Particular care was taken with the curing of the concrete for a minimum period of 14 days after depositing, the concrete being then coated with three separate applications of silicate of soda, P84 grade, to harden all the exposed surfaces.

More than 250 test cubes were made during construction, from the concrete in the stepped apron, and were crushed at an independent testing works. The average crushing strength of these cubes is shown in Table 1.

TABLE 1.—AVERAGE CRUSHING STRENGTH OF TEST CUBES

Type of cement	Average crushing strength at 7-day test : lb. per square inch	Average crushing strength at 28-day test : lb. per square inch
"417"	4,440	5,590
Rapid-hardening . . .	3,610	4,960
Normal Portland . . .	3,080	4,690

This sea defence scheme is of particular interest in that it was one of the first schemes to receive a Government grant-in-aid, following the Prime Minister's pronouncement in the House of Commons on the 27th January 1947.

The Paper is accompanied by four photographs and two sheets of drawings, from which the half-tone page plates and Folding Plates 1 and 2 have been prepared.

Paper No. 5897

"Post-War Coast Protection Works along the South-East Coast of England, which have been Undertaken by the Kent River Board as Part of their Functions in Connexion with Land Drainage"

by

George Cubley Crowther, O.B.E.

(*Ordered by the Council to be published with written discussion*) †

SYNOPSIS

The Paper is an attempt to deal with certain practical works of sea defences which have been carried out by the Kent River Board and its predecessors, the Kent Rivers Catchment Board and the Rother and Jury's Gut Catchment Board, since the end of the Second World War. In reading the Paper it should be realized that these works are, in the main, reconstructions and adaptions to existing works which have been found insufficient to meet the present-day demands for more efficient protection from the inroads of the sea.

The result is, therefore, a description of many varied types of construction, and had it not been for the existence of the original works, there is no doubt that other forms of construction might well have been adopted.

Nevertheless, since it is very seldom that complete new works are undertaken, it is hoped that the descriptions in the Paper will be of interest to many engineers who are called upon to undertake similar projects.

Introduction

The Author feels that some explanation should be given regarding the reason for a River Board carrying out coast protection works as part of its duties as a land drainage authority. It should be realized that there are large areas of land around the coast of Britain protected by sea defences of some kind which, if impaired, would result in the flooding of agricultural land and urban property and the destruction of the outfalls of many "main rivers" and arterial drainage channels. It is for this reason that the responsibility of the maintenance of such defences is placed upon the River Board as a drainage authority.

The sea defences which are the subject of this Paper are the responsibility of the Kent River Board, whose jurisdiction in this respect extends from Woolwich along the south bank of the River Thames to its eastern

† Correspondence on this Paper should be received at the Institution by the 1st June, 1953, and will be published in Part II of the Proceedings. Contributions should be limited to about 1,200 words.—Sec. I.C.E.

limit at the Isle of Grain, opposite Sheerness; the sea and the tidal-river walls in the River Medway estuary and along the north coast of Kent to Birchington; the lengths of lowland sea defences between Ramsgate and Deal and from Hythe to Hastings; in all a total length of defences of about 151 miles.

The character of these defences is very diverse—some being merely earthen embankments covered with turf or stone pitching, whilst others are heavy masonry or concrete structures built to withstand the severe storms of the English Channel. As an additional protection the Board has cultivated rice-grass plantations on certain suitable foreshores and constructed and maintained a very large number of groynes, which in themselves vary considerably in character as the nature of the foreshore demands.

The details in this Paper must necessarily be brief, but an attempt has been made to explain the various designs and materials used in the defences, and to show how these have been chosen to suit the various conditions found on the separate lengths of coast.

Many of the earthen walls were originally constructed hundreds of years ago and up to the passing of the 1930 Land Drainage Act, which set up Catchment Boards for the Rivers Medway, Stour, Romney, and Rother, the estuary walls were maintained by the Commissioners of Sewers; in fact the Romney Marsh Commissioners were the first land drainage authority ever to be brought into existence and date back to the 12th century.

The River Board Act of 1948 finally amalgamated all the above-mentioned Catchment Boards, and the present body, namely, the Kent River Board, now assumes responsibility for the whole of the defences from Woolwich to Hastings.

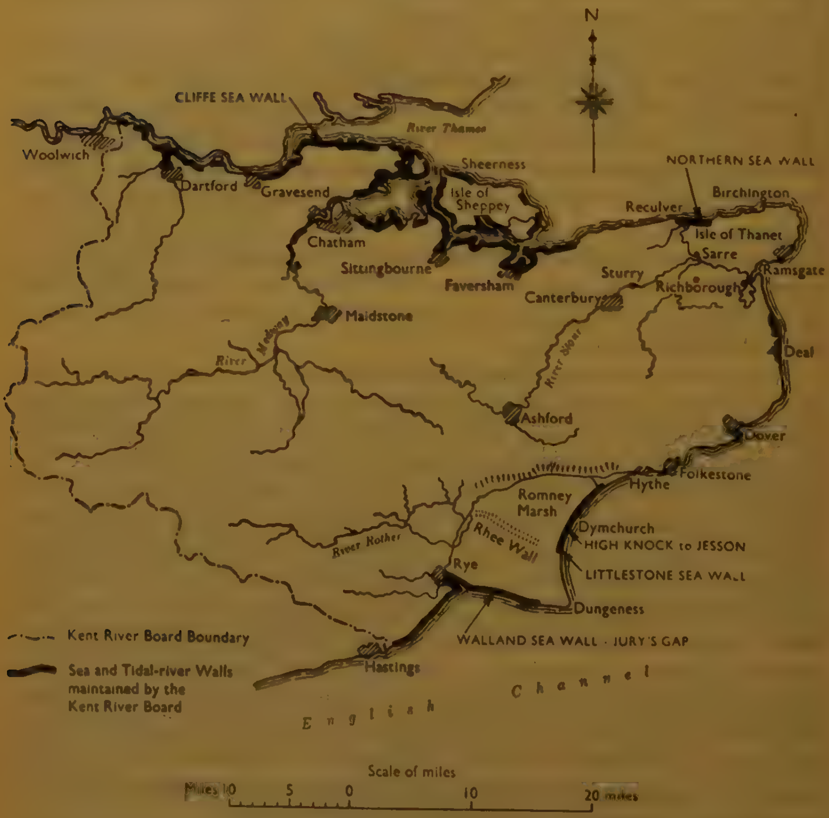
When the Catchment Boards were constituted as a result of the 1930 Act, it became evident to the engineers that very few major works had been carried out for a long time and that there was considerable leeway to be made up in the improvement and maintenance of the defences. This was no doubt because the incidence of charge upon the owners and occupiers had made the burden of these sea defences more than the lands and properties could withstand, since there was no general system of levying a charge, except upon the lands immediately adjacent; in many cases the height of the walls was below the level of the highest tides, and the top width, owing to successive raising without broadening of the base, had become almost "knife-edged"; the top layer very frequently being built-up with marsh turfs, which as time went on had become friable and very easily displaced or washed off during storm periods. The first task, therefore, of the Catchment Board and later the River Board, was to undertake work on the most urgent lengths and raise and improve the defences as quickly as possible to a reasonable standard of safety.

The intervention, first of financial restrictions, and secondly of war

conditions, severely hampered the Board in its endeavours and although the most vulnerable lengths have now been improved, the Author realizes that there is still a great deal to be done and that there remain lengths of coast where, under exceptional conditions, the risk of overtopping and damage has not yet been removed. However, the worst lengths have been put in order and at present some 120 miles of the defences now maintained by the Kent River Board have been improved and, in addition, regular maintenance works have been carried out, the latter accounting for an expenditure of approximately £62,000 per annum. The total cost of improvement works which have been carried out under the Author's direction has, to date, been £841,000.

Fig. 1 is a reduced-scale map showing the lengths of coastline along which sea defence works are maintained by the Kent River Board. It will

Fig. 1



Key Map

be seen that the map includes a length of wall south-west of Rye towards Hastings, along which considerable works have been carried out, both by the late Rother and Jury's Gut Catchment Board and the Kent River Board, but no mention is made in the Paper of the character of this work since it is understood that it is to be dealt with at a later date in a Paper by Mr C. H. Dobbie, B.Sc. (Eng.), M.I.C.E., who designed the work when he was holding the post of Engineer to the Rother and Jury's Gut Catchment Board and has since acted as Consultant for this work since the amalgamation of the Rother and Jury's Gut Catchment Board into the larger authority now vested in Kent River Board.

Earthen Sea Walls

Although many miles of earthen sea walls have been raised and strengthened by the Kent River Board, so much has been written in the past in respect of the form of walls of this nature, that little is needed in this Paper to explain the character of the work.

In the main, the walls consist of clay and loam which has been obtained from local sites, and the modern method consists in excavating a delph ditch or borrow-pit at a reasonable distance behind the wall and placing the material in position on the top and face of the original wall, properly consolidating, re-seeding with grass or turfs and, where necessary, pitching with a local stone known as Kent ragstone, which is a calcareous limestone, and if properly selected is very hard and durable. In some cases the borrow-pit forms a delph ditch and provides a drainage channel behind the wall.

In the past too little care has been given to the siting of these delph ditches; in many cases they have been dug close to the base of the wall and the modern practice is to fill in the old ditch and excavate at a suitable distance from the landward toe of the wall. Care has to be taken in the selection of material, especially along the Thames Estuary, where peat beds are encountered in layers which have to be eliminated from the material of the wall and, moreover, may become a menace to the stability of the wall itself if peat is excavated in close proximity to the new or improved embankment. Although the slopes of the wall must necessarily be the subject of much thought, the Author is of opinion that the seaward slope and the landward slope should never be less than 3 : 1 and 2 : 1 respectively. This may appear, in some cases, to be a little extravagant but in the Author's experience the additional cost in the first instance is well worth the expenditure. Not only is the wave action less severe but the longer slope prevents over-topping, with the consequent "washing out" of earthen material from the back slope which, although it is not always realized, is practically always the first to suffer damage from over-topping, and the collapse of the front face is a subsequent and final failure. The question of soil mechanics must be referred to here, since the

stability of the wall foundations has too often been neglected in the past, with the result that slips have occurred which might otherwise have been avoided.

Later in the Paper the Author will refer to various types of facing of sea walls when dealing with specific lengths, but it would not be inappropriate to say at once that in his opinion thin layers of rigid material should always be avoided. Settlement and shrinkage inevitably take place on earthen walls, and unless there is provision for flexing in the facing, cavities will develop which subsequently will cause the failure of the rigid facing and subsequent collapse. The Author has seen unknown cavities exposed by a storm, many yards in surface area, with a cavernous opening below a rigid slab which, under stress of storm, has failed and a breach has been caused in the wall. The question of flexibility in the covering of earth walls cannot be over-emphasized.

River Thames Tidal-River Wall—Cliffe to Egypt Bay

One of the works which the Board has undertaken since the war is the improvement of the length of tidal-river wall which lies to the east of Gravesend on the south bank of the River Thames, between Cliffe and Egypt Bay. The original wall consisted of a typical earthen embankment protected on the riverside by a ragstone face ; the wall was known to be low and well below extraordinary spring-tide level and the rubble stone face had disintegrated.

The new work consists in raising, strengthening, and thickening the earth wall with material obtained from the delph ditch, and the construction of a concrete block and stepped apron on the river side. The wall has been raised so that, when settlement has occurred, it will have a top level of 17·00 O.D. (Newlyn) and a top width of 6 feet, the highest recorded extraordinary tide being 15·15 O.D. Since this is a tidal-river wall and not subject to very heavy wave action, the margin of approximately 2 feet has been considered sufficient, remembering that normal high spring tide is approximately 10·24 O.D.

The form of the wall (see *Fig. 2*) is not regular—the lower length of the wall on the seaward side having a slope of 5 : 1, steepened at the top to $1\frac{1}{2}$: 1 where it is seldom washed by the tide. This steeper slope acts as a wave crest. The back slope is $1\frac{1}{2}$: 1 and the Author would have preferred that this slope should have been 2 : 1 but lack of funds made economy necessary in this respect.

It should be noted that the precast concrete blocks are interlocking and that there are two rows of steps on the lower slope to interrupt wave action.

The blocks are 15 inches square and 8 inches in depth, and the lower edge on the seaward side is sustained by a concrete toe wall. Below this toe wall the original rubble pitching has been left, since it has

Fig. 2

RIVER THAMES TIDAL-RIVER WALL—CLIFFE TO EGYPT BAY

Fig. 3 (a)

NORTHERN SEA WALL—RECULVER TO BIRCHINGTON. VIEW OF WALL DURING CONSTRUCTION

Fig. 3 (b)

NORTHERN SEA WALL—RECULVER TO BIRCHINGTON—VIEW OF COMPLETED WALL

been found that at this level, with a flat foreshore, little erosion occurs. The earth wall has been sown with a suitable mixture of grass seed to give a well-matted covering.

Attempts have been made at soil stabilization by using bitumastic material mixed with earth, but the work was expensive and as yet no decision has been made as to whether or not this method shall be continued on the front slopes. Only time will tell if this method will become economical. The cost of this scheme, which includes the raising and strengthening of 5 miles and the facing of ½ mile of earth walls, was £65,000.

Northern Sea Wall—Reculver to Birchington
(See *Figs 3*)

The sea wall between Reculver and Birchington faces north and takes the full wave-force when the winds are in a northerly and easterly direction. It is approximately 3 miles long and links the Isle of Thanet with the mainland, protecting about 12,000 acres of marshland, in addition to the main highway and railway communication between the Isle of Thanet and the remainder of Kent. The original earthen sea wall stands about 13 feet above marsh level and is protected on its seaward side by a natural and fluctuating accumulation of beach, which is retained so far as possible by timber groynes along the slope of the foreshore. In the past, the method of protection has been the periodical laying of faggot thatching along those portions where erosion has been most marked, but in the past 20 years the wall has suffered severe damage during extraordinary high-tide conditions, and in both 1938 and 1949 the wall was almost breached in several places and great anxiety was felt for the safety of the surrounding lands.

On several occasions strengthening works have been carried out by excavating material from the back of the wall, but this material consisted merely of silt with a large proportion of shingle which had been thrown over from time to time. In spite of repeated investigation no really suitable material could be found within a reasonable distance of the wall, and to transport good clay or other material from a distance was found to be expensive, owing to the fact that there was no road access, except at each end of the wall. This method was therefore abandoned in favour of some other form of construction, and it was finally decided to depart from the original method of constructing an earthen wall with a protected face. Several methods were considered and it was finally decided to construct three reinforced-concrete longitudinal walls astride the top of the wall with reinforced-concrete compartment walls so as to form monolithic structures, in 60-foot lengths. These would be superimposed on the top of the wall to form a breastwork between high-water and ordinary spring tides, and to provide a sloped apron by means of reinforced counterfort walls built on the forward slope and filled in with concrete blocks. The longitudinal and counterfort walls are 12 inches thick, and the apron is paved as stated

above with concrete blocks 15 inches square and 6 inches thick. The top of the wall forms a roadway 9 feet 3 inches wide and is coated with bituminous macadam, thus providing access for future maintenance and further construction.

The essential features of the design are the comparatively light section of the main members of the structure, which derive their strength and stability from the monolithic arrangement, and the elimination of danger from fracture caused by settlement, which is overcome by the separation of units.

The longitudinal wall at the toe of the apron slope gives protection from underscour, and the vertical wall at the top of the front face checks the wave-run and thus obviates scour of the wall on the landward side.

The height of the wall provides for a freeboard of 12 feet above mean high-tide level the level of the top surface being 21·00 O.D. The estimated cost of constructing the 3 miles of wall is £300,000, of which £23,000 has already been expended, and the work is proceeding.

Dymchurch Sea Wall and Groynes—High Knock to Jesson

The Dymchurch sea wall forms part of the sea defences for the Level of Romney Marsh, which has an area of approximately 24,000 acres and extends along the south coast from Hythe to New Romney.

The original "great wall" of Dymchurch, which is so well known, extends from Hythe to High Knock, and is a heavy earthen sea wall protected by masonry or concrete blocks on the seaward side; it has stood the test of time for many years. To construct so heavy a wall at the present day would be almost an impossibility, regarding both cost and labour, and it was therefore decided to reconsider what form of construction should be undertaken for the further length between High Knock and Jesson which lies towards New Romney.

One of the difficulties on this length was that there was no certainty of finding any other foundation save the beach and shingle, the marsh clays being very variable in level and changing in depth at quite short intervals.

Attempts were made 50 to 70 years ago to stabilize the shingle "full" by superimposing mass concrete stepwork, with an apron at the rear, and a wave wall. From time to time this stepwork has needed extensive repairs, and although there was actually no complete breach, many lengths were washed out at intervals in heavy storm periods. The Board was, therefore, faced with having either to undertake a very heavy maintenance charge or to reinstate this length of wall completely.

The Author had, therefore, to consider how the failures arose, and it is his opinion that what really happened—and does often happen in walls which are protected in this manner—was that on each rising tide the sand and shingle became charged with water (which drained out again on the fall of the tide), and this process was repeated on each and every tide, with

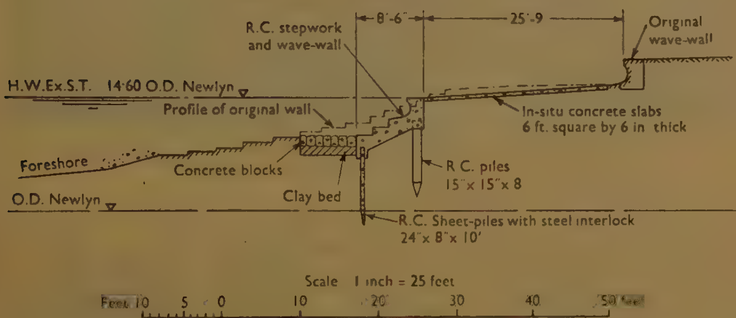
the result that over a long period fine particles were carried down and in the course of time were drawn seawards. It might be also that some of the particles were actually solvent and thus dissipated.

If, therefore, the wall is protected by a mass of concrete which depends for its strength upon base support, as soon as this latter is removed the concrete wall breaks up and eventually is completely wrecked.

The Author has frequently found hollow places under the Dymchurch apron in which a man could stand upright. When it is realized that even a small cavity, if the area is sufficiently large, will cause mass concrete to fracture, it is not surprising that such a form of construction will finally lead to its own destruction.

It was therefore decided that the middle band of the wall, which broke up more frequently, should be replaced by a form of construction which

Fig. 4



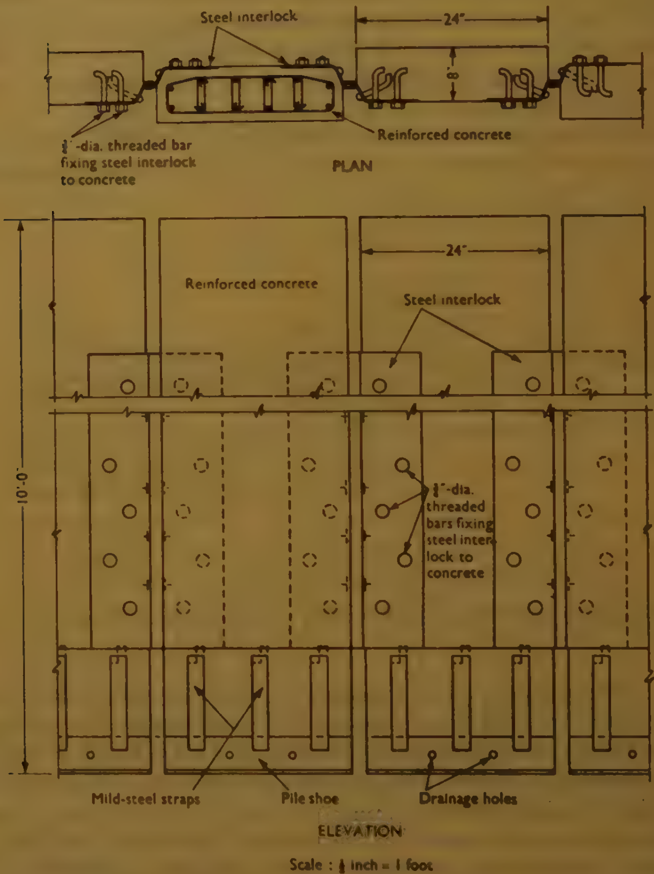
Dymchurch Sea Wall—High Knock to Jesson (*See also Fig. 6*)

would be self-supporting, and a design was eventually approved as shown in *Fig. 4*; it will be noted that a "stepped" form of middle band is shown, which is supported on concrete piles, the stepwork itself being designed as a beam so that if the earthen base support is removed the whole structure will not fail but will be self-supporting.

The front piles were considered not only to be bearing piles but also curtain piles, and these have therefore been designed of reinforced concrete into which is cast one-half of a Larssen pile, securely fixed and hooked back into the reinforcement; thus sufficient bearing strength is obtained, and also a close curtaining effect throughout the whole length. It is believed that this is the first time that this type of construction has been attempted, and although the Author was discouraged by his brother engineers in his attempt, so far as can be seen the whole design has been very successful and is carrying out satisfactorily the duties for which it was intended.

It will be noted that the design incorporates in the back of the stepwork an additional wave wall, and at the rear of the stepwork there is a flat apron which has been married into the wave crest of the existing wall; altogether the work appears to be satisfactory, both constructionally and

Figs 5



DYMCHURCH SEA WALL—HIGH KNOCK TO JESSON (*See also Fig. 6*)

(Details of R.C. Sheet-Piles with Steel Interlock)

also from the point of view that access to the foreshore is easily obtained; in addition the sea front has now become an amenity to the district; *Figs 5* and *6* show the form of the reinforced piling and also a good view of the work as it nears completion. The cost of this scheme was £215,000.

Fig. 6

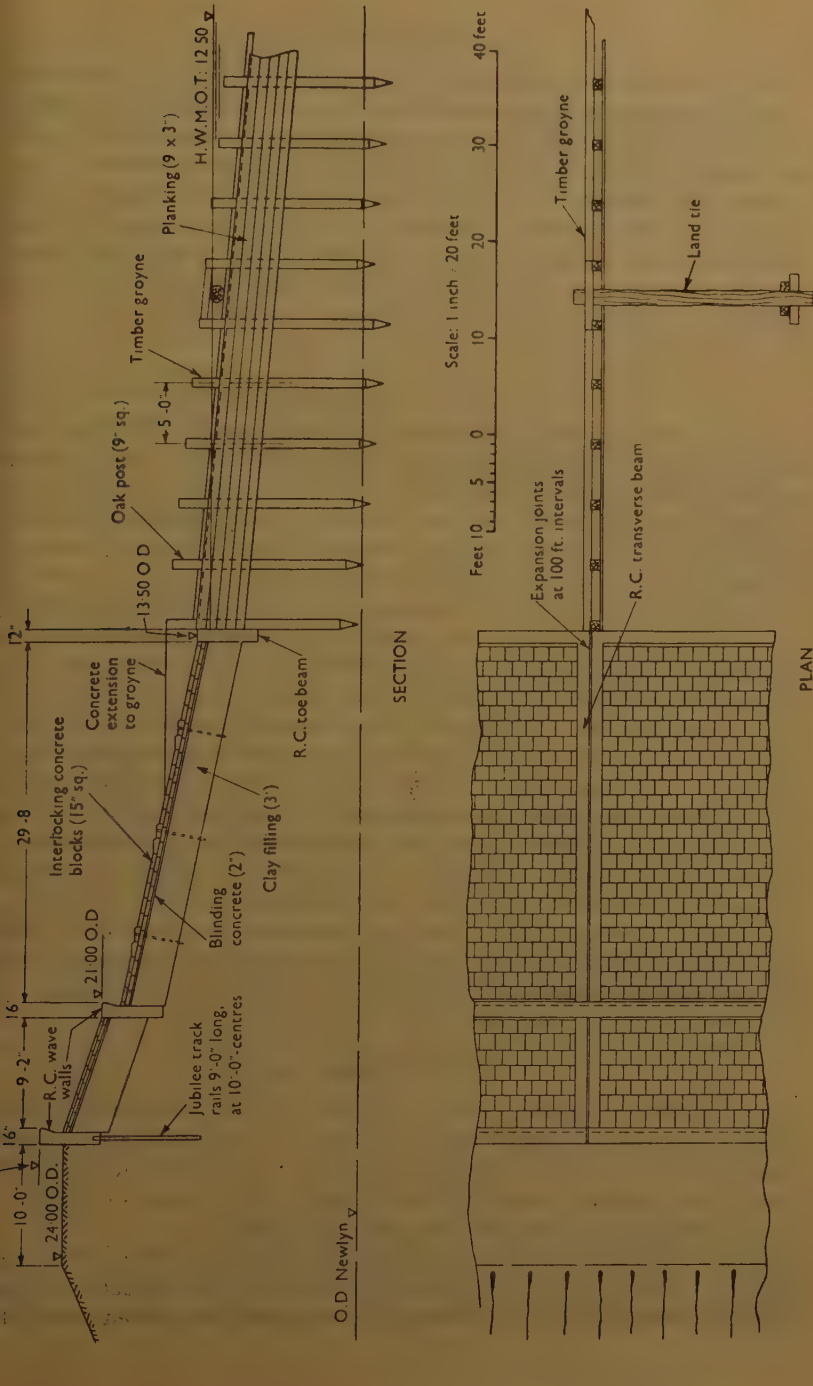
DYMCHURCH SEA WALL (*See also Figs 4 and 5*)

Fig. 7

LITTLESTONE SEA WALL

Fig. 8 (a)

WALL AND SEA WALL—LYDD TO RYE, JURY'S GAP FRONTAGE
(Upper Crest and Slope not completed)



WALL AND SEA WALL—LYDD TO RYE, JURY'S GAP FRONTAGE.

Littlestone Sea Wall

The Littlestone sea wall had been reconstructed a few years ago by the addition of stepwork supported at the toe by a row of Larssen steel piling, but it had become unstable owing to the lowering of the beach level, and it was necessary to add further protective works on the seaward side.

Work very similar to that which was carried out at Dymchurch was therefore undertaken, with the addition of a granite-block apron laid in a rectangular form between compartment walls surmounting king piles. Work in progress is shown in *Fig. 7*. The additional strengthening was completed in 1952, and the cost for 1,000 feet was £47,000.

Walland Sea Wall—Lydd to Rye, Jury's Gap Frontage

The sea wall between Lydd and Rye originally consisted of a clay wall faced by faggot thatching and, in pre-war days, so long as this type of protection was maintained annually, little or no trouble seems to have occurred. During the 1939–1945 war, however, the wall was under military jurisdiction and the faggot-work was allowed to decay, as was a series of groynes which formed the foreshore protection.

As a result of this neglect the foreshore level was lowered considerably, and it was necessary to consider what works should be undertaken to stabilize the position since the danger of breaches was becoming a menace.

The wall protects a very large area of fertile marshland and it was decided, owing to lack of labour and the expense of obtaining oak faggots, that the wall should be raised and strengthened with good-quality clay (which had to be imported), and to pave the face with interlocking precast concrete blocks laid between cross-walls so as to cut it up into sections in case of any future trouble. A deep toe wall was also constructed and the block-work was stepped. An extra wave crest was provided on the slope, as shown in *Figs 8*, which show the form of the wall.

Added to the scheme are 200-foot long timber groynes which are rooted into the apron with a concrete extension. The total estimated cost of the improvements to this wall is £148,000 and the work is now in progress.

Conclusions

The Author has endeavoured to explain, briefly, the works of sea defences which he has carried out for the Kent River Board and its predecessors, the Kent Rivers Catchment Board, since the end of World War II, but he feels it would be well to make a few remarks on his experience with regard to sea defences over many years.

First, it has become apparent to the Author that sea defences in the past have been, in the main, a matter of satisfying the exigences of the moment by means of a rapidly thought out expedient, very often executed

in great haste after some disaster has occurred, the time factor making it difficult to give complete consideration to the design. Experience of the foreshore for many years has no doubt helped engineers to devise and execute works which have given satisfaction and which have accomplished the task for which they were intended. On the other hand, little seems to be known as to the real cause of sea-defence failures which can with certainty be relied upon to help the engineer in dealing with every individual problem which he may have to attempt. Too little seems to be known of the travel of beach along the foreshores or the origin of such beach and the possibility of using it to protect the foreshore.

It is evident to any engineer that beach stabilization by the use of groynes can only be successful if the beach is travelling along the foreshore and can be intercepted, and when the limit of this interception is reached no further protection can be obtained.

The Author has found through experience that on the south coast there have been many changes in the accretion of beach which apparently are not entirely attributable to local circumstances, and in his view research on this matter is very necessary to prevent the expenditure of large sums of money on works whose success may be problematical. Very often, schemes which appeared very sound when they were conceived and executed have resulted in failures which, had there been more information available, might have been averted.

The use of models for foreshore projects can no doubt provide very useful data on general principles, but it is very doubtful whether any model can be relied upon to give the many variations, a large number of which are interrelated and occur as a result of the combination of tides, winds, and other influences which vary from time to time.

Local conditions, such as the presence or absence of deep water not too far off-shore, the growing-out of such promentaries as Dungeness, and many other features, including the operations on the flanks by other coast protection authorities, all produce complications which add to the difficulties of a maritime engineer.

The Author would suggest that the Government of the day would do well, even in these days of financial stringency, to provide funds to allow research into these problems to be undertaken, and he is certain that public funds would not suffer as a result, since most works of this character are subsidized by Government grants.

Acknowledgements

The Author wishes to acknowledge the assistance afforded him by Messrs Halskworth Wheeler of 9 Church Street, Folkestone, Kent, for their permission to reproduce the photograph of Dymchurch Sea Wall, and

to his own staff for providing the diagrams and other photographs included in the Paper.

The Paper is accompanied by six photographs and four sheets of drawings, from which the half-tone page plates and the Figures in the text have been prepared.

CORRESPONDENCE

on a Paper published in Proceedings, Part II, June 1952

Road Paper No. 37

"Recent Developments in Highway Bridge Design in Hampshire" †

by

Edwin William Henry Gifford, B.Sc. (Eng.), A.M.I.C.E.

Correspondence

Professor A. D. Ross proposed to make only a brief comment on the test.

Anomalous strains had been recorded during pre-stressing and that had been his own experience. It was plausible to add the stress distributions resulting from pre-stress and dead load but he doubted if the nominal combined distribution was ever, in fact, realized initially. Because of the usually small eccentricity of the cable at the ends of a beam, the change in curvature there was small and, although the centre lifted clear, the beam was probably supported along a considerable length at each end owing to deformations of the base and of the beam itself. In effect, the dead load was operative on a reduced span and since the moment was proportional to the square of the span the reduction in dead-load stresses could be quite marked. He believed that it was wise in design to assume that only a fraction, perhaps one-half, of the dead-load moment was acting after post-tensioning while the beam still rested on its base. When the indeterminate effects of longitudinal restraint of the base and friction of the cable were added, the resulting stresses might be very different from those envisaged by the designer, but it was encouraging to find that the strains approached their expected values when the beam was placed in the test rig.

The Author had certainly used good judgement to obtain experimentally a load factor against cracking of 1·65 in a beam with a nominal factor of 1·5. From the figures in the Paper the tensile stress in the lowest fibre due to the

† Proc. Instn Civ. Engrs, Part II, vol. 1, p. 461 (June 1952).

cracking load appeared to be $1{,}700 \times \frac{4{\cdot}95}{3{\cdot}06} = 2{,}750$ lb. per square inch. Thus, combining stresses, the following Table could be drawn up. The modulus of rupture was difficult to estimate with precision and in predicting the cracking load there was the added uncertainty of the actual loss of pre-stress which had taken place. In the case of that beam, tested early in its life, the modulus was probably in the range 600–700 lb. per square inch, a figure which agreed with Professor Ross's experience for that class of concrete. Tested after a long period, the load factor against cracking might have been somewhat less than 1·5 since the probably small increase in the modulus of rupture would not compensate for the reduction in pre-stress.

Stress due to	Initially	After relaxations
Pre-stress plus dead load	+ 2,145 lb. per square inch	+ 1,750 lb. per square inch
Cracking load (4·95 tons)	− 2,750 ,, ,, ,, ,,	− 2,750 ,, ,, ,, ,,
Nominal modulus of rupture	− 605 ,, ,, ,, ,,	− 1,000 ,, ,, ,, ,,

It was a little surprising that the ultimate load factor was only 2·40. That was the kind of figure to be expected with unbonded or poorly bonded beams, but the crack-pattern and the photographs in *Figs 30* provided ample evidence of good bonding. *Fig. 29*, however, strongly suggested that shear had been an influential factor in the final failure. Under large point loads near the centre of the span it was difficult for a heavily cracked beam to develop its full flexural strength because the greatly reduced compressive area was called upon to carry a heavy shear in addition to the high bending stresses. It seemed probable that a much higher load factor would be obtained under uniform loading since the large shears at the ends were unlikely to be troublesome because of the freedom from cracking and the beneficial inclination of the cables.

Dr K. Hajnal-Kónyi observed that it was of interest to analyse the test results given in the Appendix.

The ultimate moment was composed of the dead-load moment

$$= \frac{34^2 \times 17 \times 12 \times 1{\cdot}04 \times 12}{8} \qquad = \quad 367{,}890 \text{ lb.-inches}$$

together with the moment resulting from the weight of testing equipment plus applied load

$$= \frac{574 + (7{\cdot}23 \times 2{,}240)}{2} \times 16 \times 12 \qquad = 1{,}609{,}840 \quad ,, \quad ,,$$

Therefore, ultimate moment (max. M) $= 1{,}977{,}730$,, ,,

The design moment was composed of the moments due to the weight of beams including grout

$$= 367{,}890 \times \frac{18}{17} \qquad = \quad 389{,}530 \text{ lb.-inches}$$

and to the surfacing, together with the moment resulting from the Ministry of Transport load as assumed by the Author = 700,000 ,, ,,

Design moment, therefore, = 1,089,530 ,, ,,

The factor of safety related in conventional manner to the total design moment was 1,977,730 : 1,089,530 = 1·81. Even if the assumption for surfacing was not quite correct, the result would not be substantially affected.

The steel stress at failure was worked out on the basis of a cube strength of 8,000 lb. per square inch (as stated in the Paper), and assuming a rectangular stress block of $c_p = 0{\cdot}6 \times$ cube strength, it would be seen that:

$$\frac{\max M}{bd^2c_p} = \frac{1{,}978{,}000}{16{\cdot}5 \times 10{\cdot}375^2 \times 4{,}800} = 0{\cdot}232$$

Lever arm: depth ratio $= \frac{1}{2}(1 + \sqrt{1 - 2 \times 0{\cdot}232}) = 0{\cdot}866$

Lever arm $= 0{\cdot}866 \times 10{\cdot}375 = 9{\cdot}0$ inches

Steel stress at failure $= \dfrac{1{,}978{,}000}{9 \times 36 \times 0{\cdot}0314} = 194{,}500$ lb. per square inch.

The initial pre-stress had been $\dfrac{69{,}000}{12 \times 0{\cdot}0314} = 183{,}000$ lb. per square inch.

It would be useful if the Author could supply data on the physical properties of the wires, particularly a stress/strain diagram and data on relaxation.

With an assumed ultimate strength of 100 tons per square inch for the 0·2-inch-diameter wires (which was usual for the material available in Great Britain), the initial pre-stress was 82 per cent of the ultimate strength. That was very high and had surely caused a considerable loss by relaxation. It should be noted that the calculated stress at failure had exceeded the initial pre-stress by only 11,500 lb. per square inch, which did not indicate a great efficiency of the grout.

With the conventional method, assuming $m = 15$, the ratio of lever arm to depth would be 0·881, and with $m = 5$ (in view of the high concrete strength) that ratio would be 0·925. That second assumption would result in a lever arm of 9·58 inches and a steel stress of only 182,600 lb. per square inch, that was to say, about the same as the initial pre-stress.

On the basis of a calculated stress with rectangular stress-distribution (which gave the highest value) and with an assumed ultimate strength of 100 tons per square inch, the stress at failure represented only 87 per

cent of the ultimate strength of the material. Had the beams been pre-tensioned, there was no doubt that the ultimate strength of the wires would have been reached, or even slightly exceeded at failure, as might be predicted from many tests, both in Great Britain and abroad, since the beams were under-reinforced. A formula recently published by Guyon,[1] to which Dr Hajnal-Kónyi would refer again later, assumed a considerable excess of the ultimate strength of the wires. With pre-tensioned wires the depth of the rectangular stress block would have been $\frac{1{\cdot}13 \times 224{,}000}{16{\cdot}5 \times 4{,}800}$ $= 3{\cdot}2$ inches; the lever arm, $10{\cdot}375 - 1{\cdot}6 = 8{\cdot}775$ inches; and the moment at failure, $1{\cdot}13 \times 224{,}000 \times 8{\cdot}775 = 2{,}222{,}000$ lb.-inches.

The time lag between pre-stressing and testing to failure was not stated in the Paper. It might be inferred from *Fig. 25* that it had been between 3 and 4 weeks. If the test had been carried out at a later date (say, a year after pre-stressing) the losses resulting from the relaxation of the steel and from further shrinkage and creep of the concrete would have been greater and both the cracking and the ultimate moment would have been smaller, reducing the overall factor of safety to less than 1·8.

With pre-tensioning, the effective pre-stress in the wires after release would have hardly exceeded 65 per cent of the ultimate strength of the wires as compared with 82 per cent in the beams described by the Author, so that the loss of pre-stress by relaxation would have been less. In that case, an increased time lag between manufacture and testing would have had much less influence on the cracking moment and none on the ultimate.

Whether or not an overall factor of safety of less than 1·8 was satisfactory in pre-stressed concrete bridges was a matter of opinion, but when different systems of pre-stressing were compared from the point of view of economy, the same factor of safety against failure had to be assumed. Dr Hajnal-Kónyi wondered if the Author had taken into account the difference in ultimate load-bearing capacity between pre-tensioned and post-tensioned beams when stating, on p. 467, " that for spans from 25 feet to at least 35 feet the post-tensioned pre-cast slab section is more economical than in-situ slabs, I-beams, or composite construction."

In order to have a fair assessment of the factor of safety, it was essential to make a close approximation of the load-bearing capacity of a structure and to distinguish between pre-tensioned and post-tensioned systems. In the case of pre-tensioned under-reinforced beams the ultimate moment was practically independent of the degree of pre-stress, whilst in post-tensioned systems the ultimate moment depended primarily on the pre-stress and on the efficiency of the grout. The Author's test had shown that although " the grout had completely filled round the wires " (see *Figs 30*),

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[1] " A Study of Continuous Beams and of some Statically Redundant Systems in Prestressed Concrete." *Inst. Tech. Bât. Trav. Pub.*, Circular J No. 8 (20 Sept., 1945). See C.A.C.A. Library Translation No. 33, p. 10.

its efficiency in establishing bond between the wires and the pre-cast concrete was negligible, since an increase above the initial pre-stress as shown in the preceding calculations (that was, 5 per cent of the ultimate strength) was to be expected even with non-bonded wires.

With an assumed ultimate strength of 100 tons per square inch of the wires, the formula recently published by Guyon would give the following ultimate moment : $36 \times 0{\cdot}0314 \times 224{,}000 \times 10{\cdot}375 = 2{,}628{,}000$ lb.-inches as compared with an actual moment of 1,978,000 lb.-inches, that was to say, only about 75 per cent of the value obtained by Guyon's formula. With a smaller initial pre-stress or with the test carried out much later, the discrepancy would have been even greater.[1]

The Author's test was therefore a confirmation of Dr Hajnal-Kónyi's view, expressed elsewhere, that Guyon's formula was not applicable to the type of construction adopted by the Author.

Dr P. W. Abeles observed that the Author had mentioned two distinct approaches to pre-stressed concrete design based on (1) the "service" (working) load, and (2) the ultimate load giving a certain factor of safety. However, the design had to comply with both conditions, as was pointed out in the "First Report on Prestressed Concrete," published in 1951,[2] and as Dr Abeles had advocated for the past 10 years. That view could not be better expressed than it had been by Dr F. G. Thomas at the Conference on Pre-stressed Concrete in 1949, when he had said "(i) the load factor against ultimate failure shall be sufficient, and (ii) the deformation at working load shall be within tolerable limits."[3] The two extreme views which the Author had mentioned in the Introduction were closely associated with the two approaches. Those engineers who had been so impressed by the "novel qualities" of pre-stressed concrete "that they have failed to recognize its relationship to traditional methods and so treat it with excessive caution" most likely considered only working-load conditions and ensured a high factor of safety against cracking, assuming that to be the most important condition. A design in which ultimate-failure conditions were disregarded might result, in spite of all caution, in a brittle structure like cast iron or glass in which cracking and failure would occur simultaneously. On the other hand, engineers who considered pre-stressed concrete "merely as a further development of ordinary reinforced concrete" and "fail to make full use of its unique properties" most likely disregarded the deformation at working load.

As mentioned above, the provision of a large factor of safety against cracking might result in a brittle failure. In fact, a high factor of safety against cracking could not be combined with great resilience ; those were

[1] In his book "*Béton Précontraint*" (Paris, 1951), on p. 599, Mr Guyon attached a coefficient of 0·9 to his formula. Even so, the test load represented only about 83 per cent of Mr Guyon's value.

[2] "First Report on Pre-stressed Concrete." Instn Struct. Engrs, 1951.

[3] "Conference on Pre-stressed Concrete." Instn Civ. Engrs, 1949.

contradictory conditions. Resilience, however, was the "unique" property of pre-stressed concrete. As could be seen also from the Paper, cracks closed and became invisible on removal of loads approaching even failure conditions. Thus, a very small factor of safety against cracking would be sufficient for repeatedly occurring loads in bridge construction. Even if cracks developed owing to overloading, they would do no harm, so long as heavy impact did not occur and provided that they closed up entirely under ordinary load. That statement was based on fatigue tests [1] carried out at Liège in 1951, when cracks which had developed under special loading closed up entirely after millions of repeated loadings. On the other hand, if there was a possibility of heavy impact, as with railway underline bridges, it might be advisable to have a higher factor of safety against cracking so as to avoid cracking under an unforeseen overload (if such a case were really probable) and the possibility of gradual destruction of the bond near the cracks owing to the impact.

It would appear from the Paper that the safety factor was related to the live load only and not to the entire service (working) load, as was required for other materials and also for pre-stressed concrete.[2] Similarly, the permissible stresses appeared to relate to live load only instead of to the entire working load.

Turning to the question of the ultimate resistance of steel, Dr Abeles observed that it was interesting to note from the Paper that a good bond resistance was ensured by grouting-in the cables. However, from the test it would appear that the bond resistance was overcome just at failure. In under-reinforced sections (and the slab tested was under-reinforced), the ultimate force taken up by the steel, $T_{ult} = \frac{M_{ult}}{a}$, where a denoted the lever arm, could be taken as $K_u \cdot A_t \cdot t_{ult}$, where t_{ult} denoted the tensile strength of the steel and A_t its section area, whilst K_u represented a factor indicating to what extent the steel was used at failure. With pre-tensioned wire, K_u could be taken as unity, whilst it would appear from the test that K_u was less than unity in spite of the good bond of grout.

Dr Abeles considered that it would be interesting to hear from the Author particulars of the efficiency of transverse pre-stressing in ensuring an equivalence to a homogeneous slab structure, particularly if the transverse cables were provided at mid-depth. The case was different if separate cables were provided at the top and bottom, as shown in *Figs 19*, where transverse resistance was ensured by the steel.

Composite constructions on general lines had been excluded by the Author in his Paper except for the block solution indicated in *Figs 13*. In that case, tensioned cables were entirely embedded in the additional concrete placed between adjacent blocks. Such a solution had, in Dr

[1] P. W. Abeles, "Some New Developments in Pre-stressed Concrete." *Structural Engineer*, Oct. 1951.

[2] See ref. 2, p. 213.

Abeles's opinion, great possibilities, particularly if the pre-cast part was reduced and an in-situ topping was provided in addition to the in-situ ribs between the individual blocks. If sufficient topping was available, transverse pre-stressing was obviated, which would be a great advantage.

The use of curved members was a pleasing solution and seemed to be particularly suitable for pre-tensioning, since it was possible to provide straight wires which allowed handling from the ends with the self-weight always counteracting. However, the solution presented by the Author in which the lower cables were curved downwards seemed, in Dr Abeles's opinion, to be somewhat unsatisfactory from the point of view of ultimate load condition, when in a cracked state the steel tended to act downwards and should be anchored to the concrete as in ordinary reinforced concrete.

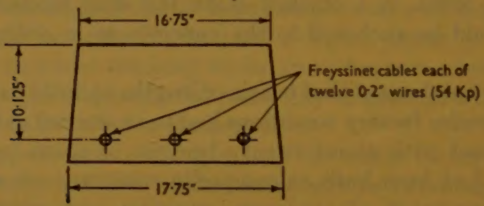
The Author's statement that beams of lengths up to 40 feet were cheaper when pre-cast under factory conditions had to be queried in the light of the experience gained with about twenty bridges, of spans between 20 and 50 feet, which had been built as composite constructions with a pre-cast pre-stressed component of minimum quantity, and which had proved to be very economical. Similarly, Dr Abeles could not understand the basis for the statement that, for spans of 25 feet to at least 35 feet, pre-cast slabs with post-tensioned cables were most economical. To his knowledge an alternative solution, consisting of a pre-tensioned girder of a weight of 6 tons and 40 feet in length, had recently been tendered at a lower price than the alternative solution with post-tensioned cables made near the site, although in the first case transport from the factory to the site and double profit of concrete works and contractor had had to be taken into account. However, it seemed to be doubtful that it was permissible to make such general statements with regard to economy, since each case had to be considered on its own merits. In any case, a composite slab, comprising a pre-stressed pre-cast component with pre-tensioned wires, costing £2 per cubic foot complete, placed in position, together with in-situ concrete costing 5*s.* per cubic foot complete, would cost only from 14*s.* 3*d.* to 16*s.* 8*d.* per cubic foot of composite slab, depending on whether the pre-cast component represented one-quarter or one-third of the entire quantity. That seemed to be less than the price per cubic foot of a pre-cast concrete slab with post-tensioned cables.

The Author, in reply, observed that the main point raised by all three contributors had been the ultimate moment of resistance. He agreed that the value obtained in the test had been slightly less than the theoretical value. He considered that the bond in the cables had been good, but agreed with Professor Ross that the system of loading might have been a possible cause of the slightly premature failure. The Author had found similar effects with two-point loadings in other tests of his own, and considered that it was not the best method of testing the resistance of a beam to bending only.

In order to clarify that point, he had decided to repeat the test on a similar beam, using a four-point loading, which would greatly reduce the chances of a shear failure. That had been done and the results for the ultimate load were given below.

It would be seen from *Fig. 31* that the beam tested had not been a precise duplicate of the previous one, the main difference being the slightly smaller lever-arm. That, however, did not affect the point in question—the efficiency of the bonds between grout and cable and between grout

Fig. 31



MID-SPAN CROSS-SECTION

Span: 34′–0″. Design live load 700,000 lb.-in.

and duct. The measure of the bond was the ultimate stress developed in the pre-stressing steel.

The applied bending moment at failure had been 1,898,000 lb.-inches.

Total bending moment was then	1,898,000
plus dead-load bending moment of beam . .	389,000
	2,289,000 lb.-inches.

The steel used in the cables had had a tested ultimate tensile strength of 111 tons per square inch, and the concrete cube strength had been about 8,000 lb. per square inch. The theoretical ultimate moment of resistance was then given by :—

Ultimate strength of three Freyssinet cables :

$$= 3 \times 12 \times 0{\cdot}0314 \times 111 \times 2{,}240$$
$$= 282{,}000 \text{ lb.}$$

Ultimate moment of resistance :

$$= 282{,}000 \times 10{\cdot}125 \left(1 - \frac{282{,}000}{2 \times 16{\cdot}75 \times 10{\cdot}125 \times 4{,}800}\right)$$
$$= 2{,}860{,}000\ (1 - 0{\cdot}172)$$
$$= 2{,}340{,}000 \text{ lb.-inches}$$

The actual value obtained in the test was therefore $\frac{2{\cdot}28}{2{\cdot}34} \times 100 = 97{\cdot}5$

per cent of the maximum theoretical value, which showed that the steel was stressed to a value approximating closely to its ultimate, and that therefore the bond had been very effective.*

The factor of safety expressed as a ratio of total bending moment to design bending moment was $\frac{2{,}289{,}000}{389{,}000 + 700{,}000} = 2{\cdot}1$ which was a value comparable with that obtained in other forms of construction; that answered the point raised by Dr Hajnal-Kónyi and Dr Abeles, namely, that it was not proper to compare the costs of two types of construction that had different ultimate strengths.

The Author agreed that composite construction could be very economical, and was in fact developing a system of construction employing that principle with post-tensioned members. However, the costs of pre-tensioned beams did depend on the amount of repetition obtainable. In the case of the railway overbridges constructed to Dr Abeles' designs, there was doubtless a great deal of repetition and it was possible that the factory overheads of manufacture had been shared by other products. The Author had since prepared designs employing the solid post-tensioned beams described in the Paper in competition with pre-tensioned composite schemes, and had found the solid beams to be cheaper.

He agreed with Dr Abeles, however, that accurate comparisons were difficult to make, since the size of the scheme and the amount of repetition had a great influence.

The method of transverse pre-stressing appeared to be effective in practice and was certainly better than depending merely on the provision of an in-situ filling on top of the beams. The Author agreed that it would be better to place the cables near to the top and bottom of the beams, and that would be done in beams of greater depth. It had not been possible in the shallow beams described in the Paper.

The amount of curvature in the beams shown had not been sufficient to require stirrups on the cables, since at ultimate load the beam was straightened out and the downward component of the cables was reduced.

* The ultimate moment of resistance obtained by Mr Guyon's formula was $282{,}000 \times 10{\cdot}125 \times 0{\cdot}9 = 2{,}570{,}000$ lb.-inches, which was a greater value than that found in the test.

It was possible that Mr Guyon had had T-sections in mind when producing that formula, since it would be much more accurate when applied to them.

CORRIGENDUM

Proceedings, Part II, October 1952

Road Paper No. 38 (Lovell, Richards, and Wilson), p. 723, 6th line from bottom.—*After* " did " *insert* " not."

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